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Journal of the
SOIL MECHANICS AND FOUNDATIONS DIVISION
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COMPUTER ANALYSIS OF SLOPE STABILITY

By John A. Horn,[1] A. M. ASCE

SYNOPSIS

A computer program has been written that can solve a great variety of slope stability problems. It is possible to analyze a slope with any surface configuration, with one or two soil strata in any pattern, and with or without water table, pore pressures, increasing cohesion with depth, tension cracks, rigid base or loads anywhere on the surface. Given these properties of a slope, the computer will automatically search out the minimum factor of safety, solving the entire problem in from one to three minutes.

STATEMENT OF THE PROBLEM

If, in a soil mass, the shear strength of the soil is exceeded, a shear failure will occur along some failure surface. In present day (1960) practice, this failure surface is commonly assumed to be, in cross section, an arc of a circle. There are an infinite number of possible failure arcs, each with a different center point, radius and factor of safety. However, the stability of the slope is governed by the minimum factor of safety. For most problems, unfortunately, it is impossible to know in advance which of all these different failure arcs has the minimum safety factor. Therefore, the factors of safety must be found for enough different center points and radii to establish a trend. After this, a conclusion can be reached as to which combination of point and radius

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gives the minimum factor of safety, and what value that minimum has. A failure condition occurs when the minimum factor of safety is less than unity.

INTRODUCTION

Undoubtedly, due to its repetitive nature, one of the most tiresome tasks that may confront the soils engineer is a complete slope stability analysis. There are numerous charts in existence which can be used to solve ideal cases. However, as we know all too well, nature rarely provides this ideal condition, the problem usually being complicated by two or more distinct soils, a water table, and so forth. It then falls upon the engineer to go through the long process of investigating different points and different radii until he finds that combination which has the minimum factor of safety.

However, it is the same repetitive characteristic which makes the problem an ideal one for presentation to a digital computer. This idea has also occurred to other people,[2,3] both in this country and abroad, and they have written programs for their own computers. No details of these works have been published as yet, but it is the hope of the author that the program and methods described here represent an advancement in this field.

The program, written for the University of Illinois computer, Illiac, is general in nature and is prepared to solve almost any practical problem presented to it.

The master program, which is punched in code on a paper tape, is read into the computer and stored in its memory. It consists of the 2000 or more instructions that tell the machine how to solve the problem. Another tape, the parameter tape, is then required to tell the computer which problem it is supposed to do. This tape, a few inches of which are alocated for each problem, contains all the data that the computer needs (soil strength, location of water table, surface configuration, and so forth). The computer reads the data for the first problem off of this tape, spends a minute or two solving it, punches the answers on another tape and immediately reads the data for the next problem, and solves it. It proceeds until all the problems contained on the parameter tape have been solved.

Although it certainly helps to do so, the user need know nothing about the program or how it works. He need only fill out a form, such as is shown in the Appendix, and get that information put on a tape.

The first section of this paper presents the theory and the equations used to compute the factor of safety. The second section gives an outline of the mechanics of the program itself.

Details of the Computer.—The Illiac, built at the University of Illinois in 1952, is an automatic electronic digital computer. It has a high speed memory of 1,024 locations and a slower drum memory of 12,800 locations. The Illiac operates at speeds several hundred times those of the so-called 'medium sized' computers. A more complete description of this computer and several photographs can be seen elsewhere.[4]

Possible Adaption of the Program to other Computers.—Since those using other computers may wish to program this problem for their own machines, a

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[2] "Electronic Computer used for Embankment Stability Analysis," by A. L. Little, London, March, 1958.

[3] "Computer Solution of the Swedish Slip Circle Method," by R. J. Hansen and R. V. LeClerc, Highway Research Abstracts, December, 1957.

[4] "Some Applications of a Digital Computer in Structural Research," by A. S. Veletsos, May, 1958.

word should be said on this possibility. The entire program requires space for approximately 2,000 orders or instructions plus 400 memory locations for numbers, that is, constants, data, temporary storage, and so forth. This fact, together with the slower speed of many computers, may make the program impossible or unfeasable for some machines. However, since the provisions for each variable (especially water table and stratification) take up both time and space, the elimination of these two should serve to shrink the program down to the size of most computers.

FEATURES OF THE PROGRAM

Before a detailed description of the program is given, it might be best to give a rough outline of what can be done by the program. Many of the features could be summed up into the statement that the program could analyze a slope,

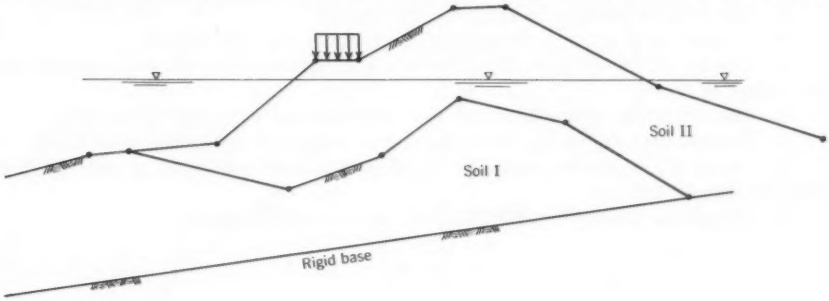

FIG. 1.—SLOPE THAT COULD BE ANALYZED BY COMPUTER

such as is shown in Fig. 1. However, for clarification, the main points will be given separately:

1. The surface and strata division lines are defined by the coordinates of up to 15 and 8 points, respectively. Both lines are specified by the user—that is, a typical slope is not built into the program.
2. A rigid base is specified by the coordinates of two points which are used to calculate a maximum radius for each point investigated. One point, that is used to compute a minimum radius, is specified.
3. A water table can be placed anywhere in the soil mass. However, it must be horizontal and at the same level both inside and outside the soil.
4. Any number of uniform distributed loads may be placed anywhere on the surface.
5. All values of cohesion and friction angle for both soil strata can be handled, except that cohesion must be less than 1,000,000 units and friction angle, ϕ, must be less than 45.
6. The cohesion may vary with depth by any c/p ratio.
7. A pore pressure coefficient ($\overline{B}$) can be specified.
8. Any consistant system of units may be assigned to the parameters-pounds and feet, tons and yards. That is, if the coordinates of the surface are expressed in meters, then the unit weight must be in weight per cubic meter, cohesion in weight per square meter.

9. Once the main program is read into the computer, it is ready to solve any number of problems successively.

10. The computer will automatically find, for each center of failure circle investigated, that radius which has the minimum factor of safety, and then, using a searching technique which investigates different center points, will find that point which has the minimum factor of safety. The point from which the computer starts its trial and error search is specified externally so that a good guess as to the location of the critical point will speed up the search process.

MATHEMATICAL FORMULATION

The assumptions are as follows:

1. All forces acting on the sides of the slices are in equilibrium.
2. The failure surface is the arc of a circle.
3. In a two soil system, both soils have the same unit weight.
4. The factor of safety is the ratio of the resisting moment to the overturning moment.
5. The resisting moment is due only to the shear strength of the soil.
6. The overturning moment is due to the weight of the entire soil mass.
7. Loads acting on the surface are carried vertically downward into the soil with no lateral effects.
8. The shear strength of the soil, S, can be expressed as

$$S = c + (p - u) \tan \phi \quad \ldots\ldots\ldots\ldots\ldots (1)$$

Calling M_R the resisting moment and M_O the overturning moment we can see from Fig. 2,

$$M_R = \int_A^B R\, s\, ds = \int_A^B R^2\, s\, d\alpha = \int_A^B [C + (p - u) \tan \phi]\, R^2\, d\alpha \ldots (2)$$

and since p, the normal stress is equal to $\gamma\ z \cos^2 \alpha$

$$M_R = \int_A^B R^2\, C\, d\alpha$$

$$+ \int_A^B \left(\gamma\ z \cos^2 \alpha\right) \tan \phi\ R^2\ d\alpha - \int_A^B u \tan \phi\, R^2\, d\alpha \ldots\ldots (3)$$

but $R \cos \alpha = Y$ and $R \cos \alpha\ d\alpha = dx$

$$R^2\ d\alpha = R\ ds = R^2\ \frac{dx}{Y}$$

then

$$M_R = \int_A^B \frac{R^2\, c}{Y}\, dx + \int_A^B \gamma \tan \phi\ Y\ z\ dx - \int_A^B \frac{u\, R^2}{Y} \tan \phi\, dx \quad \ldots\ldots (4)$$

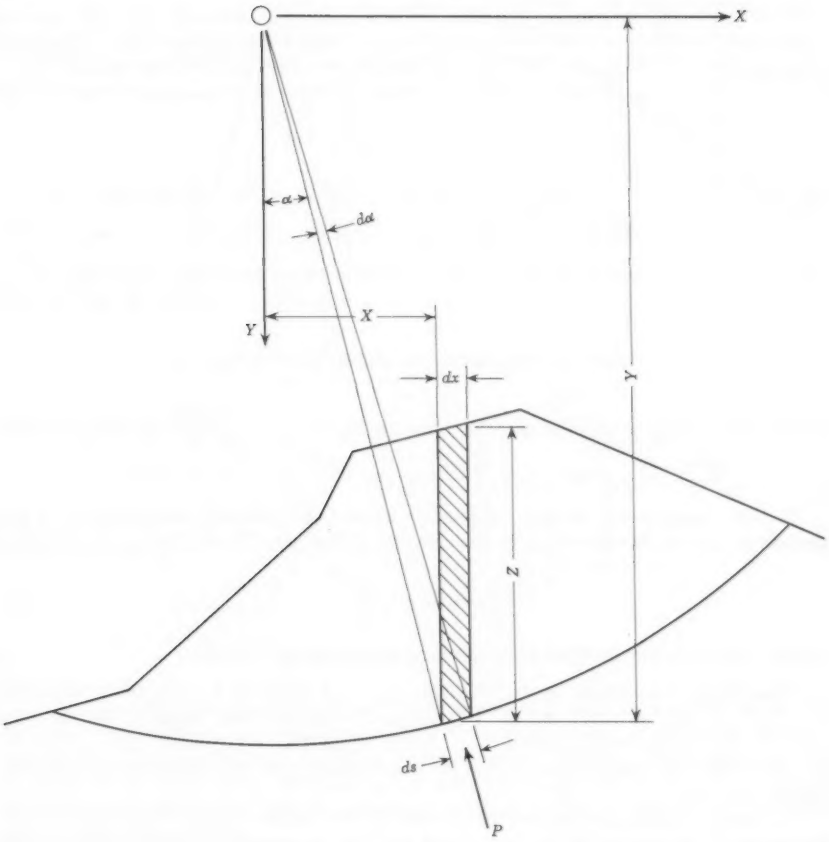

FIG. 2.—DERIVATION OF RESISTING AND OVERTURNING MOMENTS

The overturning moment M_O can easily be seen to be

$$M_O = \int_A^B \gamma \, z \, x \, dx \quad \ldots\ldots\ldots\ldots\ldots\ldots (5)$$

Most of this derivation can be credited to N. Janbu.[5] Additional variables can be sited:

1. Increasing cohesion with depth (Fig. 3)—For any point on the failure circle

$$c = c' + l \, (Y - y_O) \quad \ldots\ldots\ldots\ldots\ldots\ldots (6)$$

5 "Stability Analysis of Slopes with Dimensionless Parameters," by N. Janbu, Harvard University Soil Mechanics Series, Cambridge, 1952.

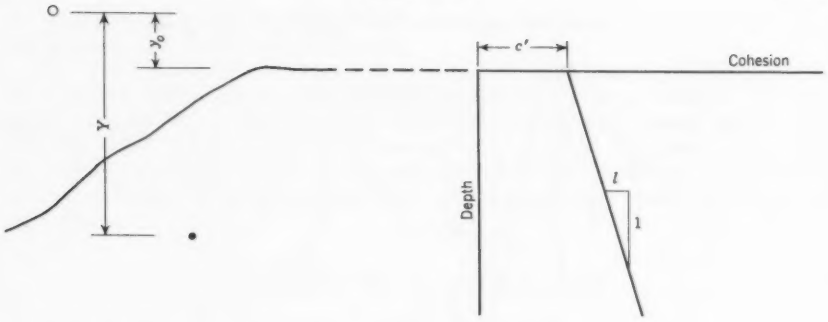

FIG. 3.—COHESION INCREASING WITH DEPTH

in which $\ell = (c/p)\,\gamma$ Therefore, the term in Eq. 4 . . . . . $\int \frac{R^2 c}{Y}\,dx$ will become . . . . . $\int \frac{R^2 c'}{Y}\,dx + \int R^2\, 1\,dx - \int \frac{R^2 1}{Y}\,y_o\,dx$

2. Surcharges—If we are willing to accept the seventh assumption of the previous listing, the problem is greatly simplified and the following can be said

$$z = z' + \frac{q}{\gamma} \quad \ldots\ldots\ldots\ldots\ldots\ldots (7)$$

which replaces the surcharge by an equivalent height of soil.

Therefore, the term in Eq. 4, M_R . . . . . $\int \gamma \tan\phi\, Y\, z\, dx$ will become . . . . . $\int \gamma \tan\phi\, Y\, z'\, dx + \int q \tan\phi\, Y\, dx$ and the term in Eq. 5, M_O . . . . . $\int \gamma\, z\, X\, dx$ will become . . . . . $\int \gamma\, z'\, X\, dx + \int q\, X\, dx$

The effect of surcharges on the pore pressure will be considered in the following section.

3. Water Table and Pore Pressures--a) No water table; The pore water pressure at any point in the soil mass can be expressed as some precentage of the vertical pressure:

$$u = \psi\,(\gamma\, z + q) \quad \ldots\ldots\ldots\ldots\ldots\ldots (8)$$

or, using Skemptons terminology:

$$\Delta\, u = B\, \Delta\, \sigma_v \quad \ldots\ldots\ldots\ldots\ldots\ldots (9)$$

During the construction period, one of the critical cases for man-made embankments, we will be dealing with Δ increments of load, so that

$$\psi = B$$

Only the resisting moment will be affected by these pore pressures, and consequently the term in Eq. 4, M_R . . . . . $\int \frac{u R^2}{Y} \tan\phi\, dx$ will become, substituting Eq. 12 in Eq. 4; . . . . . $\int \frac{\psi \gamma z R^2}{Y} \tan\phi\, dx + \int \frac{\psi q R^2}{Y} \tan\phi\, dx$

b) With a water table; As we can see, the effects of a water table would be evidenced to two areas. First, the pore pressures would decrease the shear

strength and the resisting moment. Second, the unit weight of the soil would be lowered to the bouyant unit weight, thereby effecting the overturning moment.

(1) Effect of pore pressures.—From Fig. 4, for which $h + h' = Y$, we see that the pore pressure at any point below the water table is

$$u = h\,\gamma_w = (Y - h')\,\gamma_w \quad \ldots\ldots\ldots\ldots\ldots\ldots (10)$$

Therefore, the term in Eq. 4 for the resisting moment. . . . . $\int \frac{u\,R^2}{Y} \tan\phi\, dx$ will become . . . . . $\int R^2\,\gamma_w \tan\phi\, dx - \int \frac{h'\,\gamma_w\,R^2}{Y} \tan\phi\, dx$

(2) Effect of bouyancy—The manner in which the computer program treats this is best illustrated by Fig. 5.

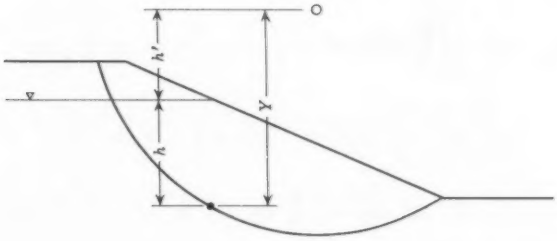

FIG. 4.—EFFECT OF PORE PRESSURE

In a computer program this is the easiest way to treat the problem since all machine instructions required to do this are also needed to compute the resisting and overturning moments. With a few changes, the same instructions can be made to do both jobs, thereby conserving space in the machine. A more complete description of how this is done is given in the second section of this paper, under Routine A.

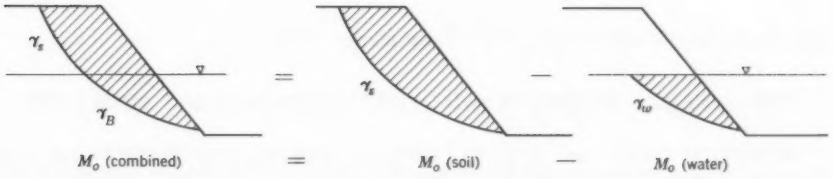

FIG. 5.—EFFECT OF BOUYANCY

Unfortunately, as was mentioned earlier, the water table must be both horizontal and at the same level within and without the soil mass. Although these specifications preclude the use of the program on many practical programs, there is no reason why the equations could not be further generalized to handle a water table of any configuration. A more sophisticated computer program can then be written, incorporating these improvements.

4. Rigid base—A rigid base will specify for each center point a radius which must not be exceeded. The maximum radius for any point would be the

perpendicular distance between that point and the rigid base. If the center of radius is also the center of the coordinate system this distance can be shown to be

$$R_{max} = \frac{(y_1 - m\ x_1)}{\sqrt{1 + m^2}} \quad \ldots\ldots\ldots\ldots\ldots (11)$$

in which X and Y are the coordinates of a point on the rigid base and m is the slope of that rigid base.

5. Tension cracks--Since a soil that contains tension cracks is generally assumed to have weight but no strength, it could either be replaced by an equivalent uniform load or, in the case of this program, it could be represented as a distinct soil strata whose shear strength is zero.

Gathering all the terms introduced with the additional variables leaves the equations

$$M_R = R^2\ c \int \frac{1}{Y}\ dx + R^2\ l \int dx - R^2\ l\ y_o \int \frac{1}{Y}\ dx$$

$$+ \gamma\ \tan\phi \int Y\ Z\ dx + \tan\phi \int q\ Y\ dx$$

$$- \psi\ \gamma\ R^2\ \tan\phi \int \frac{Z}{Y}\ dx - \psi\ R^2 \tan\phi \int \frac{q}{Y}\ dx$$

$$- \left[R^2\ \gamma_w\ \tan\phi \int dx - R^2\ \gamma_w\ h'\ \tan\phi \int \frac{1}{Y}\ dx \right] \ldots (12)$$

the final term of which is introduced only for points below water table, and

$$M_o = \gamma \int z\ x\ dx + \int q\ x\ dx$$

$$- \left[\gamma_w \int z\ x\ dx \right] \ldots\ldots\ldots\ldots\ldots (13)$$

the final term of which is $M_o^{(water)}$, as shown in Fig. 5.

THE GENERAL RESISTING AND OVERTURNING MOMENT EQUATIONS

In order to make Eqs. 12 and 13 perfectly general, they should be put in a form which can be integrated over each "constant surface slope" portion of the soil mass.

From the simple geometrical relationships, shown in Fig. 6, for which,

$$Y = \sqrt{R^2 - X^2} \quad \ldots\ldots\ldots\ldots\ldots (14)$$

$$Z = Y - y_1 - m\left(X - x_1\right) \quad \ldots\ldots\ldots\ldots\ldots (15)$$

we see that we can now express everything as a function of X if we substitute Eqs. 14 and 15 into Eqs. 12 and 13. The resulting equations will present a means of computing the resisting and overturning moments for each segment of the entire slope. When this has been done for every section the resisting and overturning moments are added up and divided to find the factor of safety.

Integrating the combination of Eqs. 12, 13, 14 and 15 we obtain first an expression for the partial resisting moment of any segment with a constant surface slope. The equation for the resisting moment M_R' for one section is

$$M_R' = R^2\ c\ \theta' + R^2\ l \left(x_j - x_i\right) - R^2\ l\ y_o\ \theta'$$
$$+ \gamma \tan\phi \left[R^2 \left(x_j - x_i\right) - \left(x_j^3 - x_i^3\right) + \frac{m}{3} \right. \quad \text{(A)}$$
$$\left. + \frac{m\ x_i - y_i}{2} \left(B + R^2\ \theta'\right) \right]$$
$$+ q \frac{\tan\phi}{2} \left[B + R^2\ \theta'\right] - \psi\ q \tan\phi\ R^2\ \theta'$$
$$- \psi\ \gamma \tan\phi\ R^2 \left[\left(x_j - x_i\right) + \left(m\ x_i - y_i\right)\theta' + m\ D \right]$$
$$- \left[R^2 \tan\phi\ \gamma_w \left(x_j - x_i\right) - R^2\ \theta'\ \tan\phi\ \gamma_w\ h \right] \quad \ldots\ldots (16)$$

for which the final term is only for section below water table and in which

$$A = \left(R^2 - x_j^2\right)\frac{3}{2} - \left(R^2 - x_i^2\right)\frac{3}{2}$$

$$B = x_j \left(R^2 - x_j^2\right)\frac{1}{2} - x_i \left(R^2 - x_i^2\right)\frac{1}{2}$$

$$\theta' = \sin^{-1}\frac{x_i}{R} - \sin^{-1}\frac{x_i}{R}$$

$$D = \left(R^2 - x_j^2\right)\frac{1}{2} - \left(R^2 - x_i^2\right)\frac{1}{2}$$

The equation for the partial overturning moment is

$$M_o' = \gamma \left[-\frac{A}{3} + \frac{m\ x_i - y_i}{2} \left(x_j^2 - x_i^2\right) \right.$$
$$\left. - \frac{m}{3} \left(x_j^3 - x_i^3\right) \right]$$
$$+ \frac{q}{2} \left(x_j^3 - x_i^3\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots (17)$$

Therefore, if we were analyzing a slope such as the one shown in Fig. 7, we would evaluate Eqs. 16 and 17 for each section. That is, evaluate them from i to m', m' to c, c to d, . . ., s' to j, with the computer automatically changing suitable parameters whenever the soil changes.

$$\text{Factor of Safety} = \frac{\sum_i^j M_R'}{\sum_i^j M_o'} \quad \ldots\ldots\ldots\ldots (18)$$

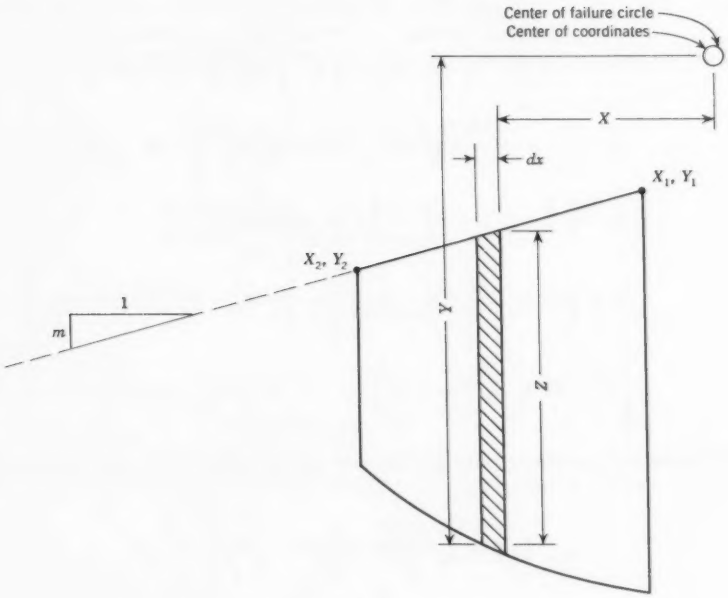

FIG. 6.—GEOMETRICAL RELATIONSHIPS FOR RESISTING AND OVERTURNING MOMENTS

METHOD OF COMPUTATION

A computer program of any size is, of necessity, composed of several groups of instructions, each group doing some special job, and each group forming one of the basic building blocks of the program. All these blocks, or routines, as they are usually called, are then tied together by the control, or master, routine which causes all the others to function as desired. This section will briefly describe these routines so as to give some idea how the program operates.

Factor of Safety for a Given Point and Radius.—This is the main building block of the program and is actually made up of several smaller blocks. This routine, called Routine A, is started with all the parameters known—in addition, it must be told by the master routine what point and radius it is to investigate.

The sequence of operation is as follows:

1. The original coordinates of the surface, water table and strata division line are transformed so that they are in a new coordinate system—one in which the origin is the center of the failure circle. This is called the transformed list.

2. Using the transformed list, the computer finds the intersections of the failure circle with each straight line portion of the surface. It then decides which two actually fall on the surface and uses these two to form a new list of coordinates. For example, in Fig. 7 first the intersections of the failure circle and line a-b are found, but since neither one lies on the surface they are both discarded. The process is then repeated for line b-c. In this case, intersection i does lie on the surface, so it is made the first point in the integration

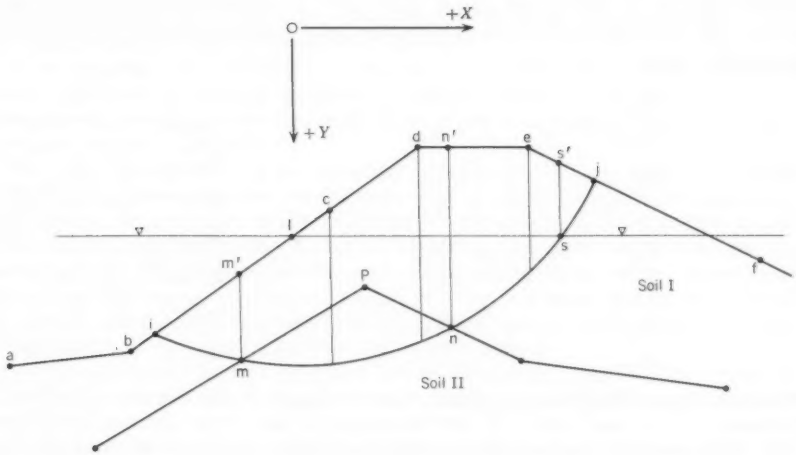

FIG. 7.—SAMPLE SLOPE

list. This goes on until a second intersection on the surface is found (in this case j). Upon completion of this routine the computer will have stored in the memory an Integration List which would consist of

X_i	Y_i		
X_c	Y_c	q_{i-c}	
X_d	Y_d	q_{c-d}	
X_e	Y_e	q_{d-e}	..... (1)
X_j	Y_j	q_{e-j}	

Note that the integration list is in reality three lists—x and y coordinates and loads.

3. Step 2. is now repeated using first the water table coordinates to form a list which, for Fig. 7, would appear as

$$\begin{matrix} X_i & Y_i \\ X_\ell & Y_\ell \\ X_s & Y_s \end{matrix} \quad \ldots\ldots\ldots\ldots \quad (2)$$

and next the strata division line to form a similar list:

$$\begin{matrix} X_m & Y_m \\ X_p & Y_p \\ X_n & Y_n \end{matrix} \quad \ldots\ldots\ldots\ldots \quad (3)$$

4. The intersection points on the circle at which some change takes place (in this case m, n, and s) are noted, and their coordinates are placed in their proper relative positions in the integration list, forming a new list called the final integration list.

$$\begin{matrix} X_i & Y_i & \\ X_{m'} & Y_{m'} & q_{i\text{-}m'} \\ X_c & Y_c & q_{m'\text{-}c} \\ X_d & Y_d & q_{c\text{-}d} \\ X_{n'} & Y_{n'} & q_{d\text{-}n'} \\ X_e & Y_e & q_{n'\text{-}e} \\ X_s & Y_s & q_{e\text{-}s} \\ X_j & Y_j & q_{s\text{-}j} \end{matrix} \quad \ldots\ldots\ldots \quad (4)$$

The bouyant water table list is ignored here since it is dealt with in another routine.

5. The computer then decides whether the failure circle in the first section (i-m) is

a. in soil I or soil II, and
b. above or below the water table.

Suitable provisions are then made depending on the answers to these questions. In Fig. 7, section i-m is in soil I and below the water table.

6. Taking into account what has been decided in step 5, the computer evaluates Eqs. 14 and 15 to find the partial resisting and overturning moments for the first section, i-m'.

7. A decision is now made as to whether the next section is in a different state than the last one. In Fig. 7, since section m'-c is in soil II, a change is made that will cause the computer to use soil II when it evaluates the resisting moment for section m'-c. Steps 6 and 7 are repeated for each section (m'-c, c-d, d-e, and e-j) until the partial resisting and overturning moments have been computed for all the sections.

8. Step 6 is repeated several more times, now using the unit weight of water instead of soil and integrating over list (2), the water table list. In this manner M_o^{wt} is computed.

9. All the partial resisting and overturning moments are now added up, the total resisting moment is divided by the total overturning moment and the factor of safety for that particular failure circle has been found.

For a slope as complicated as the one shown in Fig. 7 this entire process would take about 0.6 sec.

Since the entire problem of slope stability consists merely of finding the factor of safety for enough points to establish a trend, it is a simple matter to write a control routine which does just this. The first such routine, which uses Routine A several times, will be called Routine B.

Minimum Factor of Safety for a Given Point.—This routine investigates different radii for a given point and decides which one has the minimum factor of safety. Using the coordinates of the rigid base the machine computes a maximum radius and, similarly, uses another point to compute a minimum radius. The computer then finds the factor of safety for the maximum radius and for several other radii evenly spaced between the maximum and minimum. The lowest of all these factors of safety is noted, thereby completing the analysis of that point. The computer investigates from 6 to 12 different radii for any one point before it decides which radii has the minimum factor of safety. The time required to do this ranges from 3 to 6 sec.

It can be seen that Routine B is the control routine for Routine A since it tells A what radii it is to investigate. However, there must be another control routine in the program—one that tells B the point it is supposed to investigate. This new routine is called C.

Find the Point which has the Minimum Factor of Safety.—One obvious way of finding the point with the minimum factor of safety would be to investigate many points in a square gridwork pattern, say every 10 ft, and then decide which was the critical point. However, a much more rapid and proper method can be devised using the so-called contour lines of factor of safety. Since these contours define a sort of bowl or valley with the minimum factor of safety at the bottom, a technique is needed that will investigate a series of points, constantly moving "downhill" until the bottom is reached.

The system used in this program is illustrated in Fig. 8. The computer starts out from the point specified by the user, point a. After investigating point b and finding that it has a higher factor of safety than a, the machine would then go in the other direction, trying first point c, which is lower than a, and next point d, which is higher than c. Since the point on either side of c is higher, the computer decides that c is the lowest in that row. Returning to point c the process would be repeated, this time moving in the Y direction and investigating points e, f and g. This cycle continues until the computer finds a point that is surrounded on all four sides by higher points. This would be point f. δ is then substituted for Δ and the search is continued until a second point is found which is surrounded by higher points. The computer decides that this point has the minimum factor of safety and the problem is solved.

This method is very simple and requires little space in the computer but, as can be seen in the example in Fig. 8, it does not work too well due to two properties of the contour lines. First, the valley is long and narrow and second, the axis of the valley is inclined at about 45° from the X and Y axes. Since the points investigated are in an X, Y system, this technique rarely gets to the bottom of the valley, but instead gets hung up at one end as it did in the example. However, a third property of the contours somewhat negates this fault. This is the fact that the sides of the valley are quite steep and the bottom is very flat. Even if the location of the absolute bottom is never reached a very good minimum factor of safety is found. It has been found that, on the floor of the valley, points 30 or 40 ft apart have factors of safety differing by only 0.05.

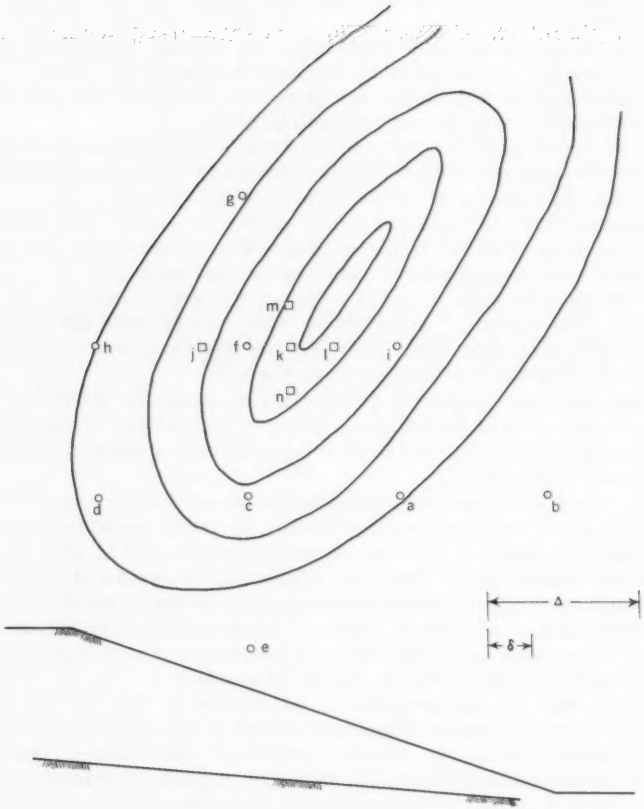

FIG. 8.—FINDING THE POINT THAT HAS THE MINIMUM FACTOR OF SAFETY

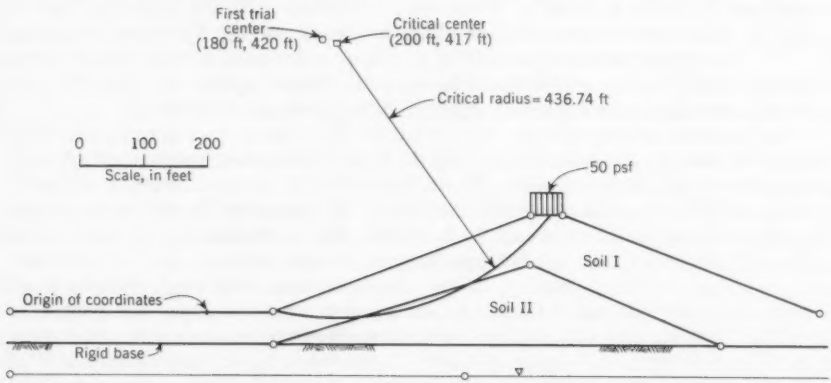

FIG. 9.—SLOPE FOR SAMPLE SOLUTION

WORK SHEET FOR SLOPE STABILITY PROGRAM

(1) SOIL PROPERTIES

Δ	δ	C_1	TAN ϕ_1	γ	C_2	TAN ϕ_2	ψ	ℓ	γ_w	W.T. HT.	y_o	
10'	3'	350 psi	0.7	145 pcf	800 psi	0.5	0	0	-62.4 pcf	-100'	150'	N

(2) X CO-ORD

	RIGID BASE	R MIN	
	+100.00	+400.00	+300.00	N

(3) Y CO-ORD

RIGID BASE		R MIN	
-50.00	-50.00	+60.00	N

SURFACE (4)	X	-300	+100	+500	+550	+950				UP TO	N
(5)	Y	0	0	+150	+150	0				15	N
(6)	LOADS	0	0	+50	0					POINTS	N

WATER TABLE (7)	X	-300	+400	+400					N
(8)	Y	-100	-100	-100					N
STRATA DIVISION LINE (9)	X	+100	+500	+800	+800				N OR J *
(10)	Y	-50	+74	-50	0				N

(11) STARTING POINT

X	Y	
+180.00'	+420.00'	N

* DOES STR. DIV. LINE INTERSECT SURFACE ?
N = NO , J = YES

FIG. 10

```
+010 +003 +000350 +7 +145 +000800 +5 +0 +0 -0624 -100 +150  N
+100 +400 +300  N
-050 -050 +060  N
-300 +100 +500 +550 +950  N
+000 +000 +150 +150 +000  N
  +00  +00  +050  +00  N
-300 +400 +400  N
-100 -100 -100  N
+100 +500 +800 +800  N
-050 +074 -050 +000  N
+180  +420  N
```

FIG. 11

CIVIL ENG DEPT SLOPE STABILITY ANALYSIS

SURFACE		LOAD	WATER TABLE		STRATA DIV LINE	
X	Y		X	Y	X	Y
-300.00	.00	000	-300.00	-100.00	100.00	-50.00
100.00	.00	000	400.00	-100.00	500.00	74.00
500.00	150.00	050	400.00	-100.00	800.00	-50.00
550.00	150.00	000			800.00	.00
950.00	.00	000				
950.00	.00	000				

SOIL I		SOIL II						
C	TAN	C	TAN	GAMMA	B	L	Y	WT HT
350.	.700	800.	.500	145	.000	.0000	150.0	-100.0

F S MIN	X	Y	R	MR	MO
1.89986	200.00	417.00	436.74	1682631304.	885237083.

FIG. 12

A much more efficient technique would be one in which the points investigated lay along lines parallel and perpendicular to the axis of the valley.

APPENDIX

A sample problem will be solved to show just what goes into the computer and what comes out. The example is largely self-explanatory but a few words might be said on the make-up of the parameter list.

1. Every number must be preceded by a sign.
2. Since all numbers must appear as fractions, the parameters must be scaled so that they obey these requirements:

- all linear dimensions are divided by 1,000.
- the unit weights are divided by 1,000.
- the cohesion is divided by 1,000,000.
- loads are divided by 1,000.
- parameters already in fractional form are left as is.

3. Decimal points do not appear but are understood to immediately follow the sign.
4. Each parameter can be represented by up to 12 digits.
5. The symbol N or J indicates the end of a set.
6. Since space limits the amount of detail which can be placed on the worksheet, Fig 10, the symbols and terms used are defined here: Δ is the initial, coarse spacing of the points investigated (see Fig. 8); δ the final point spacing (see Fig. 8); c the cohesion; tan ϕ, tangent of the friction angle; γ the unit weight of the soil; ψ the pore pressure coefficient ($\overline{B}$), ℓ the c/p ratio; γ_w unit weight of water; W.T.HT. water table height (Y coordinate of top of water table); y_o Y coordinate of the level at which the cohesion is as given on the worksheet (used only when the soil has some c/p ratio) See Fig. 3.

The slope to be analyzed is shown in Fig. 9. First, using the soil parameters, c_2 = 800 psi, tan ϕ_2 = 0.5 and γ_2 = 145 psf for Soil II and c_1 = 350 psi, tan $\phi_1 = 0.7_1$ and γ_1 = 145 psf for Soil I, and the coordinates of the points marked by circles, a worksheet such as is shown in Fig. 10 would be filled out. Then, using a teletypewriter, this data would be punched on paper tape, exactly as shown in Fig. 11. After the main program tape and the parameter tape had been read into the machine, and the problem solved, the computer automatically prints out another tape, this one containing all the information shown on Fig. 12. As can be seen, the coordinates of all the points are reprinted (the last point indicated in the water table and strata division line lists do not exist but are meaningful only to the computer). The last line gives the factor of safety as 1.89986 with the critical center point at x = 200.00 ft, y = 417.00 ft and the radius R = 436.74 ft. The resisting moment for this circle is 1,682,631,304 ft-lbs while the overturning moment is 885,237,083 ft-lbs. The accuracy displayed is, of course, practically useless. However, for the assumptions made, the answers are exact. As was mentioned before, the computer would, in this case, analyze approximately one-hundred failure circles before coming to the above conclusions and spend between two and three minutes in the process.

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FUNDAMENTAL ASPECTS OF THIXOTROPY IN SOILS

By James K. Mitchell,[1] A. M. ASCE

SYNOPSIS

Thixotropic phenomena are described and previous investigations on the behavior of thixotropic systems are summarized. The complex nature of the phenomena is pointed out, as well as the fact that thixotropy is of quite general occurrence in fine-grained materials

A hypothesis for thixotropic behavior based on initial nonequilibrium of interparticle forces after remolding or compaction, and the effects of this nonequilibrium on subsequent structure changes within the soil, is offered as one possible explanation of the phenomenon. Experimental results are presented which are consistent with the hypothesis.

Some of the practical aspects of thixotropy are pointed out, and it is suggested that thixotropic strength increases warrant consideration in design, provided they are accurately evaluated.

INTRODUCTION

The term "thixotropy" was introduced in 1927 by A. F. Peterfi (1)[2] and used by H. Freundlich (2) to describe the well-known phenomenon of isothermal, reversible gel-sol transformation in colloidal suspensions. J. M. Burgers and G. W. Scott-Blair (3) define thixotropy as a "process of softening caused by remolding, followed by a time-dependent return to the original harder state."

Note.—Discussion open until November 1, 1960. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. This paper is part of the copyrighted Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers, Vol. 86, No. SM 3, June, 1960.

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[2] Numerals in parentheses, thus (1), refer to corresponding items in the Bibliography.

Thixotropic effects in remolded natural clays have been studied by O. Moretto (4) and A. W. Skempton and R. D. Northey (5). These investigations were aimed at determining the extent to which thixotropic hardening could contribute to the sensitivity of clays. H. B. Seed and C. K. Chan (6) have shown that compacted clays may also exhibit appreciable thixotropic strength gain with time. These studies along with a study by P. G. H. Boswell (7) and conclusions reached by H. R. Kruyt (8) suggest that thixotropy may be of general occurrence in the majority of clay-water systems.

Since thixotropic effects can result in strength increases of up to 100% or more after remolding, even in compacted soils, and thus could be of significance from an engineering standpoint, it is useful to examine the phenomenon in some detail. A knowledge of the factors responsible for thixotropic strength increase should aid in further understanding of soil structure and shear strength behavior.

At the present time thixotropy implies slightly different things to different investigators. For example, to the soil engineer, thixotropy signifies strength increase with time after remolding or compaction. He is concerned with water contents generally less than the liquid limit and times measured in weeks, months, or even years. On the other hand to the colloid chemist or rheologist thixotropy immediately brings to mind dilute suspensions and gel setting times of minutes, hours, or, at most, a few days. In fact it has been suggested that the term thixotropy should not be applied to the phenomena of time-dependent strength gain in soils and a term such as "age hardening" be used. If however, thixotropy is defined as an isothermal, reversible, time-dependent process occurring under conditions of constant composition and volume whereby a material stiffens while at rest and softens or liquifies upon remolding, then the difference between thixotropy in fine-grained soils at water contents less than the liquid limit (concentrated suspensions) and dilute suspensions may be considered one of degree rather than fundamental behavior. The properties of a purely thixotropic material are illustrated in Fig. 1.

In the following two sections, literature that sheds some light on the fundamentals of thixotropy is reviewed. Next, a hypothesis for the cause of thixotropic behavior is presented. Finally, experimental datum is offered which is consistent with the hypothesis.

THIXOTROPY IN DILUTE SUSPENSIONS

Although dilute suspensions bear little resemblance to clay soils in terms of physical properties, it is not unreasonable to suppose that some of the interparticle force mechanisms and principles of aggregation and dispersion are the same as in more concentrated clay-water systems. Freundlich (2) was among the first to present a detailed treatise on the subject of thixotropy. He pointed out that thixotropic behavior undoubtedly depends on the balance of forces acting between particles. The great influence of electrolyte concentration in the system is noted, with an increase in electrolyte content leading to a decrease in setting time for a thixotropic gel.

A tendency for particles of the same substance to adhere upon contact may be important in thixotropy All particles of finely divided matter exhibit an unbalance of forces at their surface. These unsatisfied forces can be satisfied either by contact with particles of the same substance or by adsorption of ions from the adjacent phase. It is possible that particles in a suspension, as

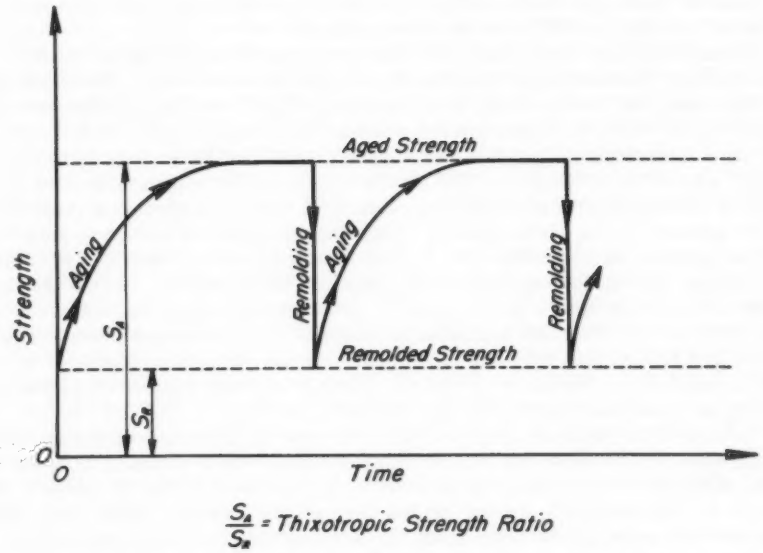


FIG. 1.—PROPERTIES OF A PURELY THIXOTROPIC MATERIAL.

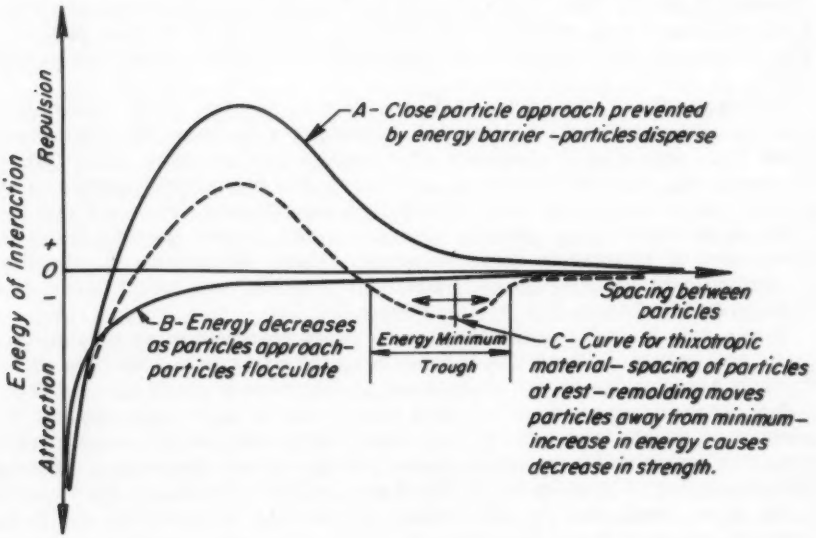

Repulsion
Energy of Interaction
+
0
-
Attraction
A- Close particle approach prevented by energy barrier -particles disperse
B- Energy decreases as particles approach- particles flocculate
Energy Minimum
Trough
Spacing between particles
C- Curve for thixotropic material- spacing of particles at rest- remolding moves particles away from minimum- increase in energy causes decrease in strength.

FIG. 2.—ENERGY-DISTANCE CURVES FOR DILUTE SUSPENSIONS OF DISPERSED, FIOCCULATEDAND THIXOTROPIC MATERIALS.

a result of their thermal motion, may be brought into such positions that they adhere, causing solidification of the suspension.

Freundlich also concluded that thixotropy cannot be explained without acknowledging the effects of far-reaching forces of attraction. The explanation of thixotropy on the basis of attraction-repulsion force balance has received wide support by other investigators, such as C. E. Marshall (9), E. J. W. Verwey and J. T. Overbeek (10), and T. W. Lambe (11). This concept is illustrated in Fig. 2 where the ordinates to the curves represent the energy (positive if repulsion and negative if attraction) necessary to bring the particles from an infinite spacing to any given spacing along the abscissa. Curve A represents a stable suspension which exhibits neither flocculation nor thixotropy because of the energy barrier preventing close approach of particles. Curve B represents a condition where particles will spontaneously agglomerate and settle out. The energy minimum indicated on curve C represents the position of particles in a thixotropic gel. Any movement (such as caused by shaking or shearing strains) which tends to change the particle spacing causes an increase in energy of repulsion leading to a more fluid condition.

Curves of the form of those in Fig 2 are usually derived on the assumption of parallel plate-shaped particles charged only on their surface. Lambe(12), J. K. Mitchell (13), H. Van Olphen (14), R. K. Schofield and H. R. Samson (15), and P. A. Theissen (16), among others have shown, however, that clay particles deviate quite appreciably from the assumptions on which the curves are derived. An objection to the situation illustrated in Fig. 2 arises from a consideration of particle orientation in thixotropic systems. According to Van Olphen (14), and R. Fahn, A. Weiss, and H. Hoffman (17), particles in the gelled state are linked in a random array, forming somewhat of a rigid structure. Fig. 2 assumes particles sitting at some distance from each other in a parallel array.

Marshall (9) and Van Olphen (14) present evidence that many times there exist two separate zones of thixotropic behavior for the same material. It was noted from studies of the behavior of suspensions of constant concentration, but increasing electrolyte content, that reversible hardening—liquifaction effects occurred first at the very low salt contents. Slightly higher salt contents led to stable dispersions. Further increase in salt content resulted in the reappearance of thixotropic characteristics. These two regions of observed thixotropic behavior correspond to zones of "non-salt" and "salt" flocculation as described by Lambe (12) and Van Olphen (14).

In non-salt flocculation particles presumably associate in an edge-to-face array, as a result of attractions between negative surfaces and positive edges. Particle associations in salt-flocculated systems are a combination of face-to-face, edge-to-edge and edge-to-face arrangements, that result when the depression of double layers due to increased electrolyte content is sufficiently great that repulsive forces are no longer able to prevent flocculation. Recent light-scattering studies by M. B. Mac Ewen and M. I. Pratt (18) have shown, on the other hand, that for very dilute suspensions of bentonite, the basic structural unit is a linear aggregate one lattice layer thick, in which particles are oriented edge-to-edge in the form of flat ribbons. No evidence of an edge-to-face or face-to-face association was found. Additional studies by Mac Ewen and Mould (19) led to the conclusion that these structures form in zones of potential minima. When the bentonite is dispersed in water the particles are flocculated with edges in close contact; whereas, in the presence of alkali the particles are associated in a minimum such as indicated by the energy trough

for curve C in Fig. 2. In this case the edges are separated by a potential barrier preventing close approach, but there is no indication of a parallel face-to-face arrangement. Regardless of the type of particle association there seems ample evidence that thixotropic behavior is associated with a tendency for particles to flocculate, as long as they are free to choose their positions.

H. L. Roeder (20) studied the rheological behavior of thixotropic systems and suggested that thixotropy occurs in systems composed of unstable or partly unstable particles. A system may then be visualized in which the particles have sufficient repulsion forces to resist immediate aggregation but will fall victims to such tendencies in the course of time.

Kruyt (8) points out that thixotropy occurs preferentially, although perhaps not exclusively, in systems with elongated particles, such as clays. The phenomena may be explained by a certain slowness of gel formation, based on either the analogue of slow flocculation or on the rareness of particle encounters due to slow thermal motion of the particles. There may also exit cases where particles are held by long-range forces. Kruyt also noted that thixotropic suspensions of bentonite have been observed wherein the particles come to rest although there is no material contact between them. On the other hand a suspension of graphite in mineral oil is nonconducting when fluid but conducting after gelation, indicating that particles in this system must be in contact and form a continuous network.

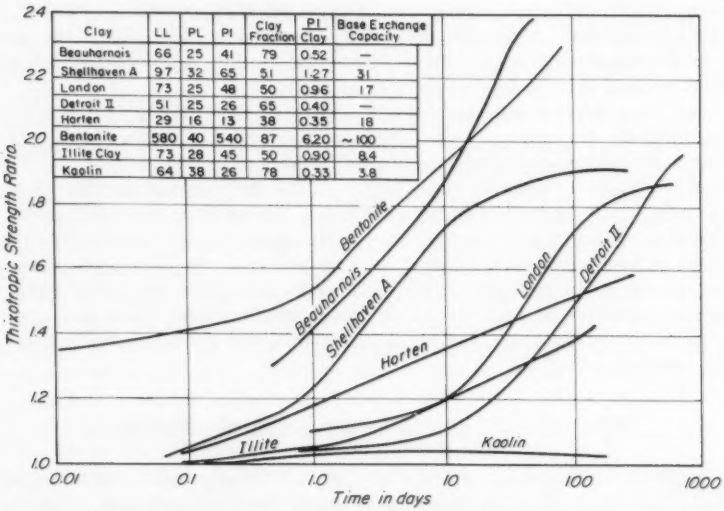
THIXOTROPY IN CONCENTRATED SUSPENSIONS

Moretto (4), Skempton and Northey (5), L. Berger and J. Gnaedinger (21), and Seed and Chan (6) all present data which vividly illustrate the increase in strength of clay soils resulting from aging at constant water content. It is apparent from the work of these investigators that this hardening can be of importance in a great variety of soils at water contents of concern to the engineer (plastic limit to liquid limit). Seed and Chan (6) also showed that the hardening effect was completely reversible for a compacted silty clay. Thus the age-hardening phenomena apparently has the characteristics of thixotropy as previously defined. However the mechanism of thixotropic hardening has not been explained. Fig. 3 summarizes the strength gain data as obtained by the above named investigators.

The thixotropic strength ratio, that is, the strength at time t divided by the strength at time equals zero) as suggested by Seed and Chan (6) rather than absolute strength increase is used as a measure of thixotropic effects in order to permit comparisons between soils of different composition, or between samples of the same soil at different water contents. In these cases the absolute strength increase might be misleading. For example a sample with a remolded strength of 1 kg per sq cm might show a strength increase to 1.1 kg per sq cm after a certain time, thus having a thixotropic strength ratio of 1.1. A second sample might have an initial strength of 0.1 kg per sq cm and increase to 0.2 kg per sq cm in the same time. The thixotropic strength ratio for this case would be 2.0. It is felt that for a material to double its strength (even though its initial strength is low) is more significant in terms of thixotropic effects than for a sample to increase its strength by 10%, even though the actual strength increase for the two materials is the same.

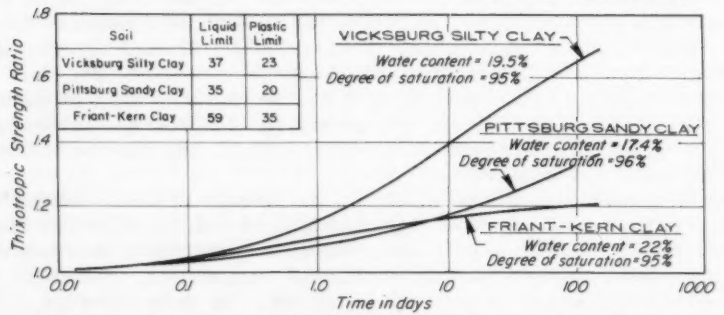
Examination of Fig. 3 reveals several points of interest. With the exception of kaolin all the soils showed a significant strength increase with time of aging.

However, tests at the University of California have shown that even kaolinite may be made very thixotropic by the addition of a dispersing agent in order to reduce the degree of flocculation present in the natural material. There appears from Fig. 3 to be no unique relationship between thixotropic strength ratio and time.


Clay | LL | PL | PI | Clay Fraction | PI / Clay | Base Exchange Capacity
Beauharnois | 66 | 25 | 41 | 79 | 0.52 | —
Shellhaven A | 97 | 32 | 65 | 51 | 1.27 | 31
London | 73 | 25 | 48 | 50 | 0.96 | 17
Detroit II | 51 | 25 | 26 | 65 | 0.40 | —
Horten | 29 | 16 | 13 | 38 | 0.35 | 18
Bentonite | 580 | 40 | 540 | 87 | 6.20 | ~100
Illite Clay | 73 | 28 | 45 | 50 | 0.90 | 8.4
Kaolin | 64 | 38 | 26 | 78 | 0.33 | 3.8
Thixotropic Strength Ratio
Time in days
Bentonite
Beauharnois
Shellhaven A
London
Detroit II
Horten
Illite
Kaolin

(a) Thixotropic strength gain at water contents equal to the liquid limit.

(Data from A.W. Skempton & R.D. Northey)


Soil | Liquid Limit | Plastic Limit
Vicksburg Silty Clay | 37 | 23
Pittsburg Sandy Clay | 35 | 20
Friant-Kern Clay | 59 | 35
VICKSBURG SILTY CLAY
Water content = 19.5%
Degree of saturation = 95%
PITTSBURG SANDY CLAY
Water content = 17.4%
Degree of saturation = 96%
FRIANT-KERN CLAY
Water content = 22%
Degree of saturation = 95%
Thixotropic Strength Ratio
Time in days

(b) Thixotropic strength gain at water contents approximately equal to the plastic limit.

FIG. 3.—SUMMARY OF THIXOTROPIC STRENGTH GAIN DATA.

The curves in Fig. 3 for the samples tested at water contents near the plastic limit show that thixotropy can be significant at low water contents. In other cases it may be negligible as shown by G. A. Leonards (22), who found no thixotropic tendencies in two compacted soils. The role of water content in determining thixotropic behavior will be considered in more detail subsequently.

Available data all indicate that the lower the strain the more pronounced is the thixotropic effect. This effect is indicated in Fig. 4 from Seed and Chan (6)

which shows the thixotropic strength ratio after 1 week as a function of water content for a compacted silty clay. Optimum water content for the compaction procedure used was 17.5%. From Fig. 4 it is seen that the thixotropic effect is small for samples dry of optimum and the effect is the same for all strains. On the other hand, for samples prepared wet of optimum the thixotropy is very pronounced, particularly at low strains. These samples were prepared by kneading compaction. It is believed that a flocculated structure is induced dry of optimum which changes gradually to a dispersed structure wet of optimum by this method of compaction. (See Seed and Chan (23) and Lambe (12) for a complete discussion of the types of structure induced in compacted soils by various methods of compaction and the effect of these structures on properties.

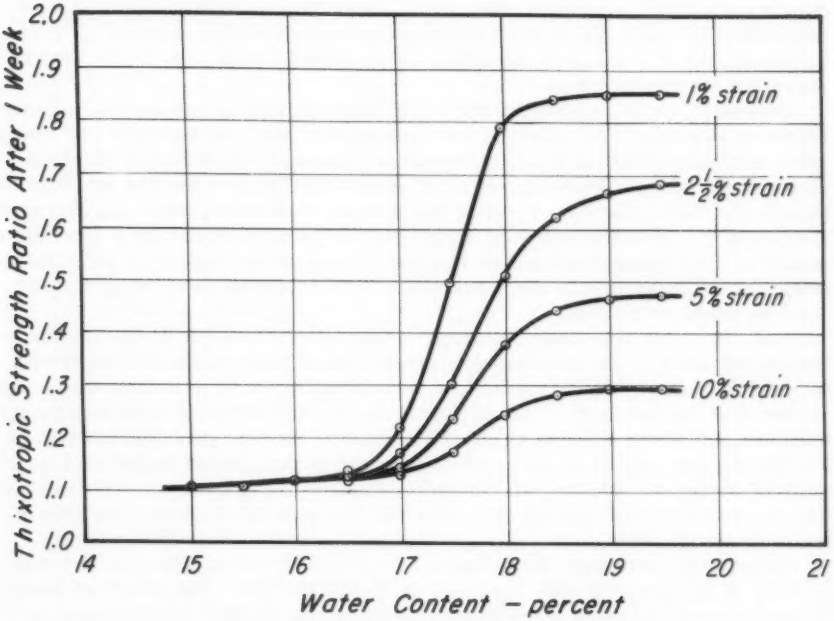


FIG. 4.—THIXOTROPIC STRENGTH RATIOS FOR DIFFERENT VALUES OF AXIAL STRAIN.

The term flocculated structure refers to a structure in which clay particles are arranged in a more or less random array and where interparticle forces were predominantly attractive at the time the soil was compacted. The term dispersed structure refers to a structure in which clay particles are arranged in a more or less parallel array and where interparticle forces were predominantly repulsive at the time the soil was compacted.) The possibility therefore exists that thixotropy depends to some extent on initial structure as well as initial water content. The decrease in thixotropic effect at high strains is to be expencted since shearing tends to remold the soil.

Data obtained by Berger and Gnaedinger (21) indicate that aging may lead to an increase in stiffness without any consequent increase in ultimate strength. It thus becomes apparent that thixotropic effects must be referenced to some

criterion of measurement, such as a particular strain, in order to evaluate their true significance.

The influence of thixotropic effects on clay compressibility is not apparent from a study of available literature. It would seem reasonable that if the strength of a soil increases with time after remolding then it might also undergo less volume change, at least under small pressures, during consolidation than if no thixotropic stiffening took place, because strength and resistance to compression both depend in some way on the strength of interparticle bonds.

Berger and Gnaedinger (21) performed consolidation tests on a plastic clay and noted no significant differences in compression curves for samples tested at 0, 6 and 12 months after remolding. On the other hand, P. L. Newland and B. H. Allely (24) found that a sensitivity was developed in a highly plastic clay during consolidation under a given load. After thorough remolding of the consolidated clay and reapplication of the same consolidation pressure, a further decrease in void ratio was observed. It is possible that thixotropy was responsible for this behavior.

Boswell (7) made an examination of the thixotropy of a great number of sedimentary deposits. All the materials examined, with the exception of clean sands, exhibited thixotropic characteristics. Boswell noted further that many of the soils were rheopectic; that is, gentle motion accelerated the rate of thixotropic hardening. In many of the soils at high void ratio syneresis was also noted; the gels begin to split open along irregular cracks with a liberation of water. This phenomenon can be attributed to a contraction of the solid phase structure, due possibly to the formation of more and more interparticle attractive bonds with time.

D. H. Trollope and Chan (25) hypothesize that thixotropic strength gain in compacted soils is due to changes in disposition of the colloidal particles within the soil mass, since according to their hypothesis of soil structure, any variation in the strength of the colloidal matrix in a constant stress environment can only occur if the mean effective distance between particles is changed. These changes, it is suggested, are accomplished through the redistribution of internal stresses. Internal stress redistributions occur because it is unlikely that the most stable colloidal structure will be immediately-established everywhere on compaction.

Perhaps the most significant datum available thus far relating to the fundamentals of thixotropy is that reported by P. R. Day (26). The effect of shear on the forces of cohesion between clay particles was studied through measurements of the intensity of the water adsorptive forces. A measure of these forces was obtained by determination of the soil moisture tension (negative pore pressure) using a simple tensiometer. Saturated clay samples at water contents near the liquid limit were allowed to come to equilibrium with the tensiometer; the suction was noted and then the sample was stirred with a motor driven mixer. Changes in tension with time after stirring were noted. Fig. 5, which summarizes some of Day's results, indicates that tension falls to a minimum value after stirring and then gradually returns to nearly the initial value, at which point the process could be repeated. Although no strength values were obtained during these tests, it was noted that the "stirring action produced a decrease in viscosity and that the gel slowly returned to its original viscous state as the tension was restored." Fig. 5 shows that the phenomenon of decreased tension (or increased pore pressure) occurred in a variety of different clay materials, and therefore appears to be of quite general occurrence.

The fact that aging a thixotropic material leads to increased water tension (decreased pore pressure) aids in the fundamental understanding of the phenomena. It is to be expected that an increase in tension, and thus an increase in effective pressure, would lead to an increase in strength. It is the basic cause of this increase in tension that is of interest. According to L. D. Baver (27)

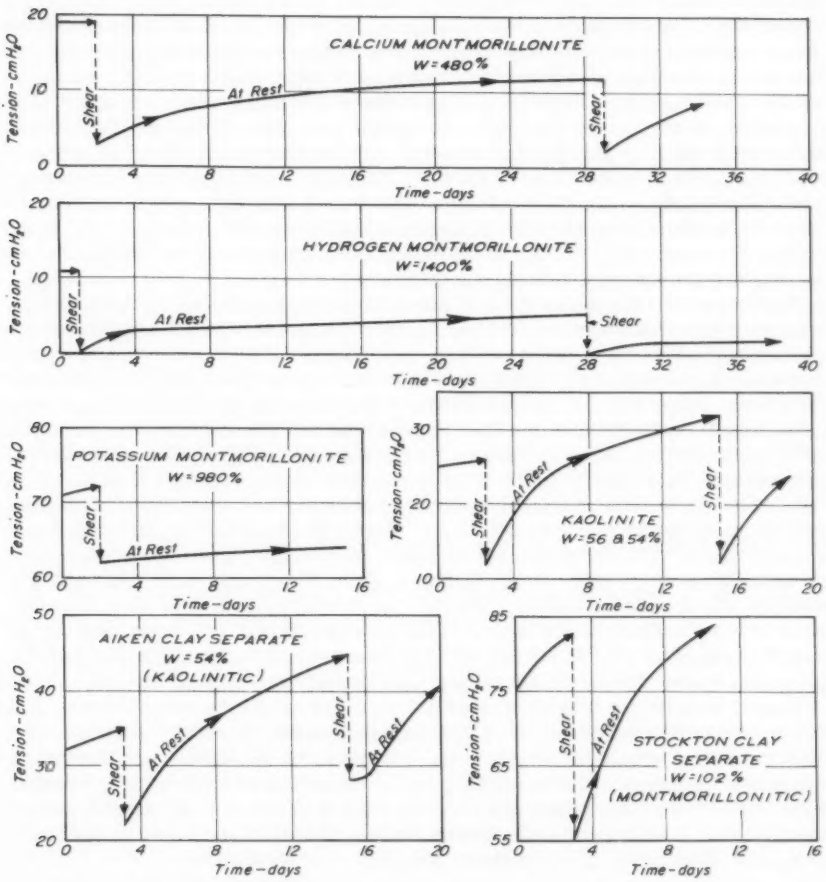


FIG. 5.—THE EFFECT OF SHEAR ON PORE WATER TENSION FOR VARIOUS CLAYS.

the tension in the soil water is a function of four factors (assuming the soil is not subjected to external forces).

1. The total hydrostatic head in the water;
2. The capillary potential due to air-water interfaces;
3. Osmotic pressure differences arising from ionic distributions in the double layers between particles; and
4. Adsorptive forces due to the attraction of water molecules to particle surfaces.

In tests on saturated clays conducted under identical conditions the first two factors are constant, thus time and shear dependent changes in the osmotic or swelling pressure and in water adsorptive forces at particle surfaces become of interest. Although the adsorptive forces at the surfaces of the clay particles may remain constant regardless of whether or not the material is at rest or undergoing stirring, the structural arrangement of the water molecules in response to these forces could conceivably differ. This would be reflected through the tension values measured. Any changes in particle orientation would involve changes in the amounts of double layer interaction between particles which in turn would be reflected through the observed tensions.

That the tension or negative pore pressure increases with time after stirring is of great importance since at constant volume the tension is a direct measure of the free energy of the water. An increase in tension (decrease in pore pressure) is associated with a decrease in free energy and a decrease in tension (increase in pore pressure) is associated with an increase in free energy. Thus, direct evidence is available to indicate that the free energy of the system decreases with time, which means that spontaneous reactions are occurring and the system is seeking equilibrium.

Finally, actual chemical change, for example, time-dependent formation of cementation bonds between particles, must be considered as a possible cause of thixotropic behavior. Iron oxides, carbonates, organic matter, and amorphous silica and alumina are known to exist in varying quantities in most soils. All of these materials could bond particles together. Remolding or compacting a soil could destroy these bonds. Whether or not these bonds would reform with time is not known. The observed reversibility of thixotropic systems argues against the formation of additional cementing materials, particularly in soils which have been in equilibrium with the same water for long periods of time or in dilute suspensions which gel almost instantaneously after shaking. It is not meant to imply however, that chemical changes cannot take place during aging of a moist soil. A. C. Mathers, S. B. Weed, and N. T. Coleman (28) have presented data to show the partial conversion of hydrogen montmorillonoids to the aluminum form in several days when stored both moist and dry at normal temperatures. R. T. Martin (29) found that lithium kaolinite converts to aluminum kaolinite on aging for a period of 100 days in 70 C water.

Thus it may be reasonable to conclude that chemical changes may occur in thixotropic systems and assist strength gain tendencies. There may also be thixotropic systems in which chemical changes are negligible. It should be pointed out, however, that according to the definition of thixotropy presented at the beginning of this paper the process occurs under conditions of constant composition. Therefore any effects due to changes of the type found by Mathers, et al., and Martin would not, strictly speaking, be thixotropic.

A HYPOTHESIS FOR THE CAUSE OF THIXOTROPIC BEHAVIOR

The following hypothesis is offered as one possible explanation of thixotropic effects in soils. The hypothesis is based on the assumption that the internal energy and stress conditions in a thixotropic soil immediately after remolding are not equilibrium conditions. This assumption appears reasonable since, if equilibrium conditions existed, there would be no changes in properties with time. But properties do change, and Day's results (26) show that the material undergoes an energy decrease, which is compatible with spontaneous changes.

This hypothesis is intended to apply to those soils in which thixotropy is primarily a structural effect; that is, property changes with time result from changes in such factors as particle arrangement, adsorbed water structure and distribution of ions in the fluid phase. Any chemical effects must be considered in addition to the mechanism outlined in the following paragraphs.

The structure assumed by a given soil is in direct response to its composition (including pore fluid type and amount) and the environmental conditions at the time of formation, plus changes in structure arising during its history. Such factors as electrolyte content and type, temperature, pH, and subsequent pressures are determinant for sedimented soils. In the case of compacted soils the method of compaction is extremely important. Lambe (12) and Seed and Chan (23) have shown the important role of structure in determining compacted soil properties. Seed and Chan have also convincingly demonstrated the important influence of method of compaction in determining soil structure (23). Furthermore, it has been suggested (12) and effectively shown (23) that the effect of shear strain on a soil is to disperse, that is, to force a more or less parallel arrangement of particles in the shear zone.

Most soils have been found to have a balance of interparticle forces leading, at equilibrium, to a structure somewhere between complete dispersion and complete flocculation. That is, few soils have been encountered that cannot be either further dispersed or further aggregated by appropriate mechanical or chemical treatment. The ease with which such structure change may be accomplished, of course, varies widely for different soils, and at different water contents for the same soil. That a given soil exhibits different thixotropic properties at different water contents has been previously pointed out.

Based on these observations, it may be postulated that a soil will exhibit thixotropic behavior if the following conditions are met:

1. The net interparticle force balance is such that the soil will flocculate if given a change.
2. The flocculation tendency is not strong enough, however, that it cannot be overcome by mechanical action, that is, shearing or stirring the soil.

The mechanism may thus be visualized as follows. When a thixotropic soil is remolded or compacted a part of the externally applied shearing energy is utilized in dispersing the platy clay particles into a uniform, parallel arrangement; that is, the externally applied energy assists the repulsive forces between particles resulting from double layer interaction in producing a dispersed system. The energy of interaction between particles is at a level comcensurate with the externally applied forces, and the adsorbed water layers and ions are distributed in accordance with this high energy level. The net result is a structure similar to that shown schematically in Fig. 6(a). Such a structure is compatible with the conditions during the shearing process.

However, as soon as shearing ceases the externally applied energy (a part of which had been assisting the normal interparticle repulsive forces in creating a disperse structure) drops to zero. Thus the net repulsive force decreases; or, stated another way, the attractive forces now exceed the repulsive forces for the particular arrangement of particles and distribution of water. As a consequence the structure attempts to adjust itself to a new, lower energy condition. The energy dissipation may be accompanied by changes in particle arrangements, adsorbed water structure and distribution of ions. Since structural changes of this type are dependent on actual physical movement of particles, water and ions, they are time-dependent. These internal structural changes

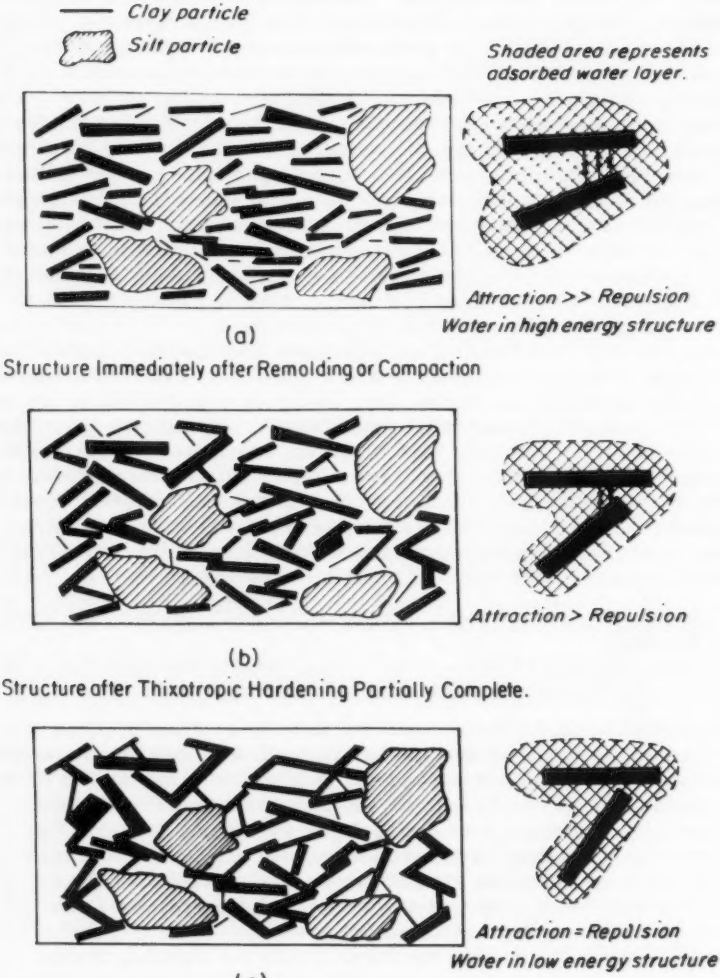


FIG. 6.—SCHEMATIC DIAGRAM OF THIXOTROPIC STRUCTURE CHANGE IN A FINE GRAINED SOIL.

are, in turn, reflected by the physical behavior of the soil (such as increasing strength).

Curves depicting energy of attraction, energy of repulsion, and total energy of interaction between particles for the condition at rest and during remolding are shown in Fig. 7. Although these curves cannot be constructed on a quantitative basis, they are useful qualitatively for the purpose of indicating the energy status within a thixotropic soil. In Fig. 7(a) are shown curves for double layer energy of repulsion and energy of attraction. In addition a line is shown indicating an additional energy of repulsion introduced by the externally applied shearing strains. This energy is shown as increasing with decreasing particle spacing because it is felt that the smaller the particle spacing the greater the percentage of the externally applied energy required to initiate particle movement. At large particle spacings a greater percentage of the applied energy is simply converted to kinetic energy of particle motion. The curve for net energy of interaction is humped, indicating an energy barrier preventing close approach or flocculation of particles. Thus a weak, dispersed structure is formed. When the external strains cease to be applied, however, Fig. 7(b), the energy of repulsion is decreased leading to a total energy of interaction as shown. The energy barrier preventing flocculation is removed.

The system may be visualized as having an excess of internal energy for the new conditions. A spontaneous dissipation of this energy occurs. Since, in the case where attraction exceeds repulsion, minimum energy demands that particles flocculate, the particles will attempt to do so. If this is to happen, however, local redistributions of ions in the double layer and of the double layer water itself are required. Concurrently with these processes a slightly altered adsorbed water structure will form. Since these processes require a dispalcement of particles and a movement of ions and water they are not instantaneous. A viscous flow of water is involved which requires time. Thus the excess energy dissipation is not instantaneous, but occurs over protracted periods. As a result, property changes continue to occur for appreciable periods. A schematic diagram of the structure at some intermediate time after remolding is shown in Fig. 6(b) the final structure is shown in Fig. 6(c). The time required for the achievement of equilibrium will vary with the initial difference between the "as-molded" structure and the equilibrium structure and the mobility of the particles which is related to such factors as water content, particle size distribution, particle shape, the ease of displacement of adsorbed water molecules, and the magnitude of effective stress during aging.

Since the process occurs at constant volume the particle movements during thixotropic hardening are probably small and of a rotational nature. A difference of a few Angstroms spacing between points of closest particle approach could have a pronounced effect on strength, however, since interparticle attractions vary as a negative power of the spacing and only small decreases in spacings at points of closest particle approach could lead to large increases in strength.

Changes in structure of the adsorbed water itself could be a significant contributor to thixotropic effects. That the free energy of the water is diminished with time after remolding is clearly shown by the tension measurements of Day (26) and additional data presented subsequently. Whether this energy decrease is caused by an actual change in the arrangement of water molecules adjacent to the clay surfaces in response to the removal of the remolding energy, or whether it reflects the changes in particle arrangement and spacing is not known. Quite possibly it reflects both factors.

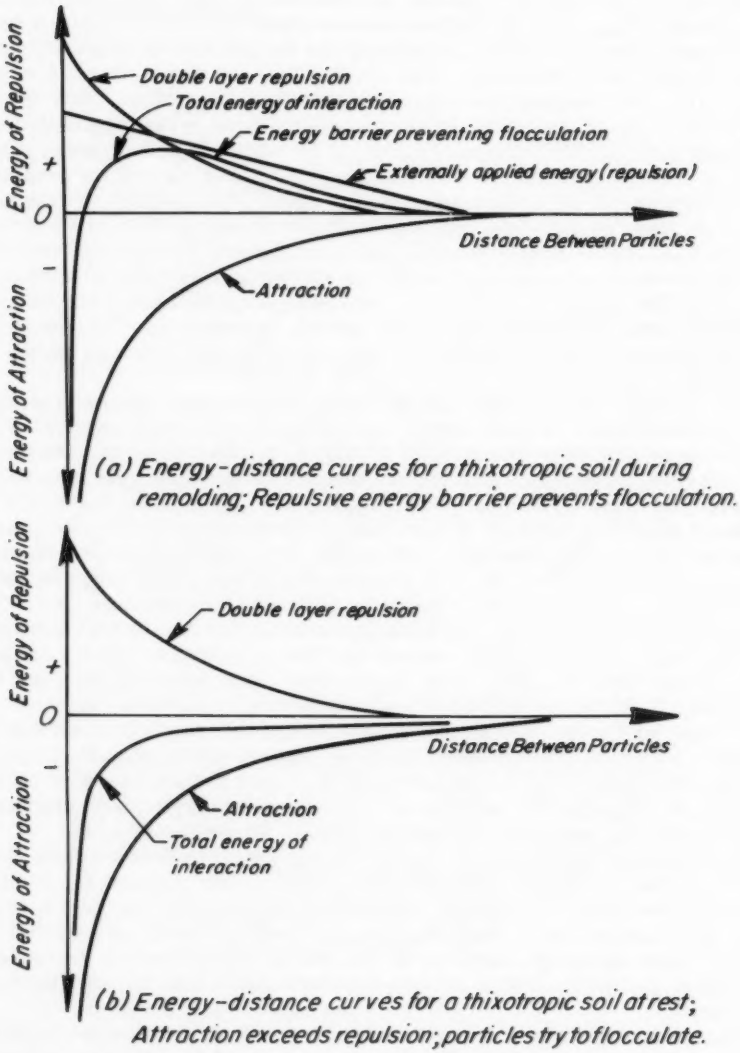


FIG. 7.—ENERGY-DISTANCE CURVES FOR A THIXOTROPIC SOIL.

In summary, thixotropic behavior is the natural response of a soil structure to a change in ambient conditions. When a soil is remolded or compacted a structure is induced which is compatible with the externally applied shearing stresses. When shearing stops the soil is left with an excess of internal energy which is dissipated by means of small particle movements and water redistribution until a structure in equilibrium with the at rest forces is created. Observed behavior of thixotropic soils is considered in the light of this hypothesis in the following sections.

EXPERIMENTAL RESULTS

Reversibility.—It is well known that the sol-gel transformation of dilute thixotropic suspensions is completely reversible. Whenever shaking or stirring of the suspension ceases the material gels; when stirring again commences it liquifies. Since the definition of thixotropy as proposed states that the phenomenon is completely reversible, it is desirable to consider the extent of reversibility at water contents of non practical interest to the engineer. The results of a reversibility study on a silty clay soil from Vicksburg, Miss. (LL = 39%, PL = 25%) are shown in Fig. 8. The aged strength of this compacted silty clay soil after 6 days storage is indicated on the figure and varied from 1.1 to 1.8 times the original as-molded strength, depending on the water content and the strain at which the strength was measured. Values of stress required to cause a given amount of strain as a function of molding water content for samples tested immediately after compaction and for samples aged 6 days, remolded, recompacted, and tested are shown. The data for the as-compacted and the remolded aged samples show no significant variation. It may be concluded, therefore, that the stiffening process was reversible. Additional evidence of complete reversibility is indicated in a later section in connection with the effect of temperature on thixotropy.

The Effect of Initial Structure on the Thixotropic Behavior of Compacted Clays.—If thixotropic effects are caused by dispersion-flocculation phenomena, then it would be expected that the relationship of the structure of a soil immediately after compaction to the structure that would exist if all particles were free to choose their own positions would influence the magnitude of thixotropic property changes. In the case where interparticle forces are such that particles are strongly flocculated and remain flocculated in spite of applied external strains then an initially flocculated structure will result. Negligible thixotropic effects would be expected since the initial structure is one of equilibrium. Similarly if interparticle forces are such that dispersion is the equilibrium condition then applied, external strains will have little additional effect and the soil should show negligible thixotropic behavior. However, if the interparticle forces dictate flocculation, but weak flocculation, and the strains induced during compaction are able to disperse the structure, then, with time after compaction, the structure will seek equilibrium through reflocculation. In this case thixotropic effects should be pronounced.

Vicksburg silty clay which shows rather unique structure changes as a function of water content was investigated. This material when compacted dry of optimum exhibits a flocculated structure, regardless of the method of compaction used, as shown by Seed and Chan (23). At water contents greater than optimum the structure may be dispersed by means of a large shear strain inducing method of compaction (such as kneading) but remains flocculated for non-shear strain inducing methods of compaction (such as static). At considerably

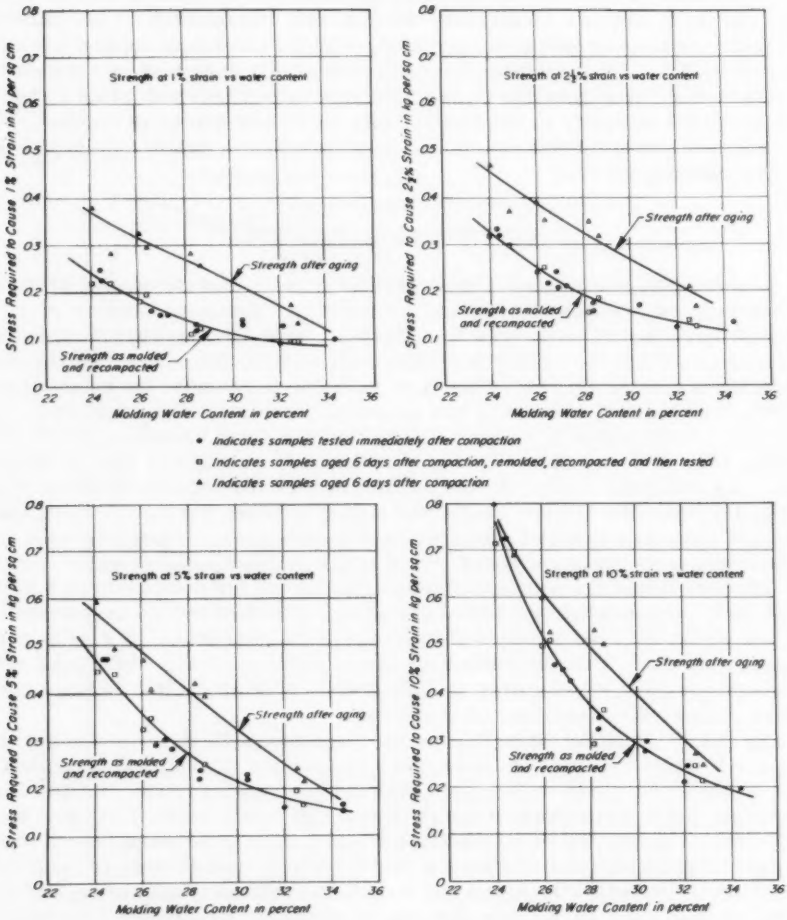

Strength at 1% strain vs water content
Stress Required to Cause 1% Strain in kg per sq cm
Strength after aging
Strength as molded and recompacted
Molding Water Content in percent
Strength at 2½% strain vs water content
Stress Required to Cause 2½% Strain in kg per sq cm
Strength after aging
Strength as molded and recompacted
Molding Water Content in percent
• Indicates samples tested immediately after compaction
□ Indicates samples aged 6 days after compaction, remolded, recompacted and then tested
▲ Indicates samples aged 6 days after compaction
Strength at 5% strain vs water content
Stress Required to Cause 5% Strain in kg per sq cm
Strength after aging
Strength as molded and recompacted
Molding Water Content in percent
Strength at 10% strain vs water content
Stress Required to Cause 10% Strain in kg per sq cm
Strength after aging
Strength as molded and recompacted
Molding Water Content in percent

FIG. 8.—COMPARISON OF THE AS-COMPACTED AND THE AGED-REMOLDED-RECOMPACTED STRENGTH OF VICKSBURG SILTY CLAY.

higher water contents this material disperses of its own accord. Wet mechanical analyses of this soil gave identical results with and without a dispersing agent. It thus seems likely that at low water contents the material is strongly flocculated due to double layer water deficiency; a condition described by Lambe (12). At high water contents the material disperses of its own accord from high double layer repulsions. At intermediate water contents the structure may be made to disperse through the application of shear strains. It would therefore be expected that thixotropic effects in this soil would be negligible at low and high water contents and pass through a maximum at some intermediate water content.

Samples were prepared by kneading compaction over the range of water contents from 14% to 35%. Optimum water content for this material is about 17.7% for the compactive effort used. Curves indicating the deviator stress required to cause 1%, 2-1/2%, 5% and 10% strain in triaxial compression tests (quick tests with σ_3 = 1.0 kg per sq cm) as a function of molding water content are shown in Fig. 9, for samples tested immediately after compaction and for samples tested after aging for 6 days. Compaction data for these samples are shown in Fig. 9(e). It may be noted from Fig. 9 that for all strains the curves for both the aged and non-aged samples are essentially parallel up to a water content of about 22%, at which stage they begin to diverge. From 22% to about 29% water content the strength of the aged samples decreases at a slower rate with increasing water content than does the strength of the non-aged samples. At water contents above 29% the strength of the aged samples decreases at a faster rate with increasing water content than does the strength of the non-aged samples. At 34% water, which was the practical upper limit of water content for triaxial testing, the strengths of both types of samples are approximately the same. It is apparent that aging had the greatest effect on strength in the range of water contents of 22% to 29%. It is also evident from Fig. 9 that the aging effect was greatest when measured at low strains.

The thixotropic effect is illustrated more graphically in Fig. 10 which was prepared from the data in Fig. 9. Here, the thixotropic strength ratio (that is, the ratio of the strength of the aged sample to the strength of the non-aged sample for the particular strain under consideration) is plotted as a function of water content. The effect of strain is also readily apparent. These results are consistent with the concepts previously presented: strong flocculation at low water contents, dispersion at high water contents and a weak state of flocculation susceptible to dispersion by shear at intermediate water contents. A decrease in thixotropic effect with increasing strain during shear is to be expected since the strains progressively destroy the thixotropic structure that has formed on aging.

These data also clarify the role of water content in thixotropic behavior. Water content appears to be of primary importance in determining the status of interparticle forces. If a soil can be made to pass from a state of flocculation to dispersion as water content is increased then thixotropic effects will be noted. Thixotropic effects have been found to increase with increasing water content in most past investigations probably because either a sufficiently high water content was not studied to permit partial dispersion under shear strain or the soil remained flocculated regardless of the water content used. However, results reported by Skempton and Northey (5) indicate that for five different clays aged 100 days three show an increase in thixotropy with increasing water content over the range studied; whereas two pass through a maximum at or near the liquid limit and then drop off for further water content increase.

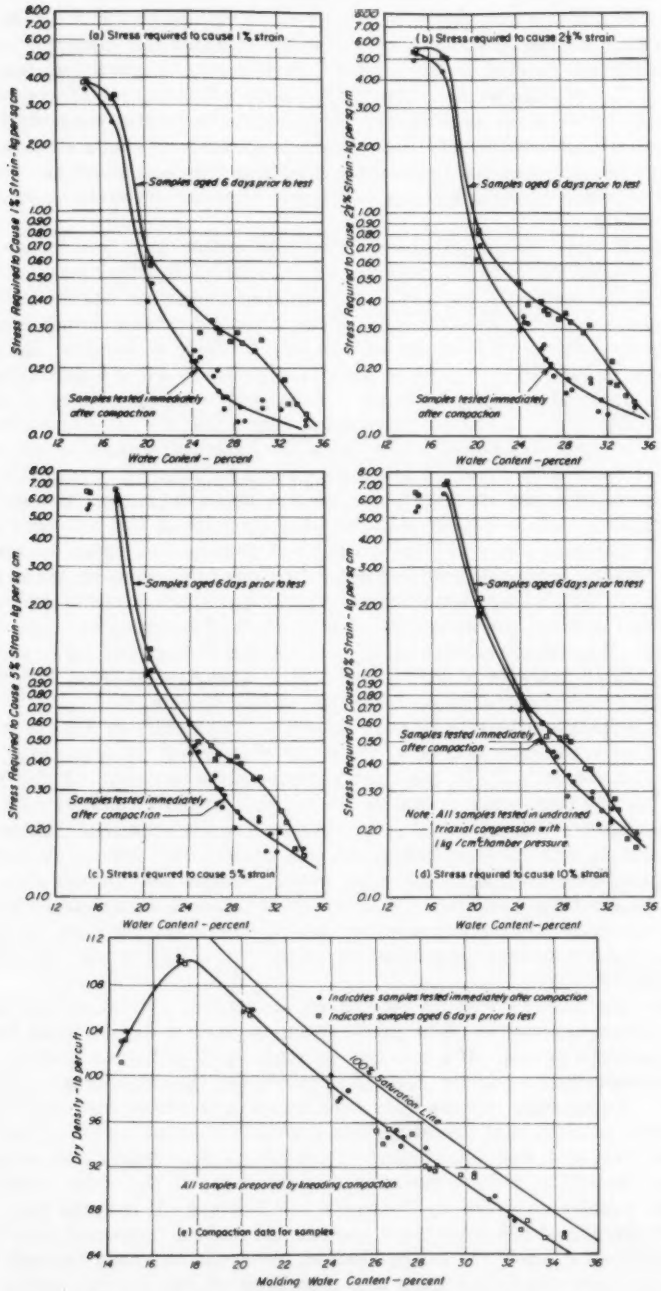

(a) Stress required to cause 1 % strain
Stress Required to Cause 1% Strain - kg per sq cm
Samples aged 6 days prior to test
Samples tested immediately after compaction
Water Content - percent
(b) Stress required to cause 2½% strain
Stress Required to Cause 2½% Strain - kg per sq cm
Samples aged 6 days prior to test
Samples tested immediately after compaction
Water Content - percent
(c) Stress required to cause 5% strain
Stress Required to Cause 5% Strain - kg per sq cm
Samples aged 6 days prior to test
Samples tested immediately after compaction
Water Content - percent
(d) Stress required to cause 10% strain
Stress Required to Cause 10% Strain - kg per sq cm
Samples aged 6 days prior to test
Samples tested immediately after compaction
Note: All samples tested in undrained triaxial compression with 1 kg/cm² chamber pressure
Water Content - percent
(e) Compaction data for samples
Dry Density - lb per cu ft
Indicates samples tested immediately after compaction
Indicates samples aged 6 days prior to test
100% Saturation Line
All samples prepared by kneading compaction
Molding Water Content - percent

FIG. 9.—STRENGTH OF COMPACTED VICKSBURG SILTY CLAY AS A FUNCTION OF MOLDING WATER CONTENT.

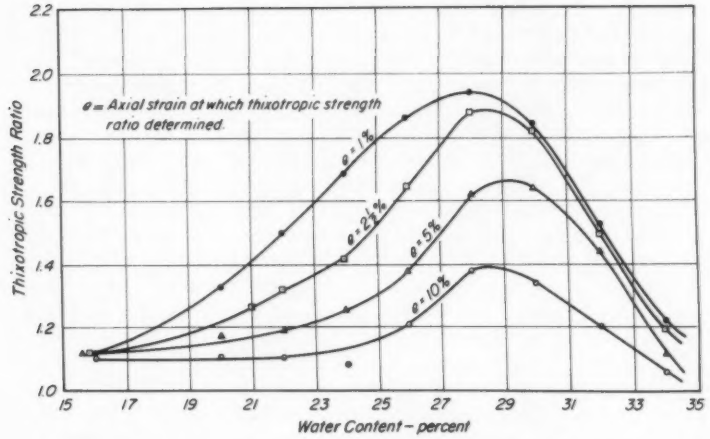


FIG. 10.—THIXOTROPIC STRENGTH RATIO FOR SILTY CLAY AS A FUNCTION OF MOLDING WATER CONTENT.

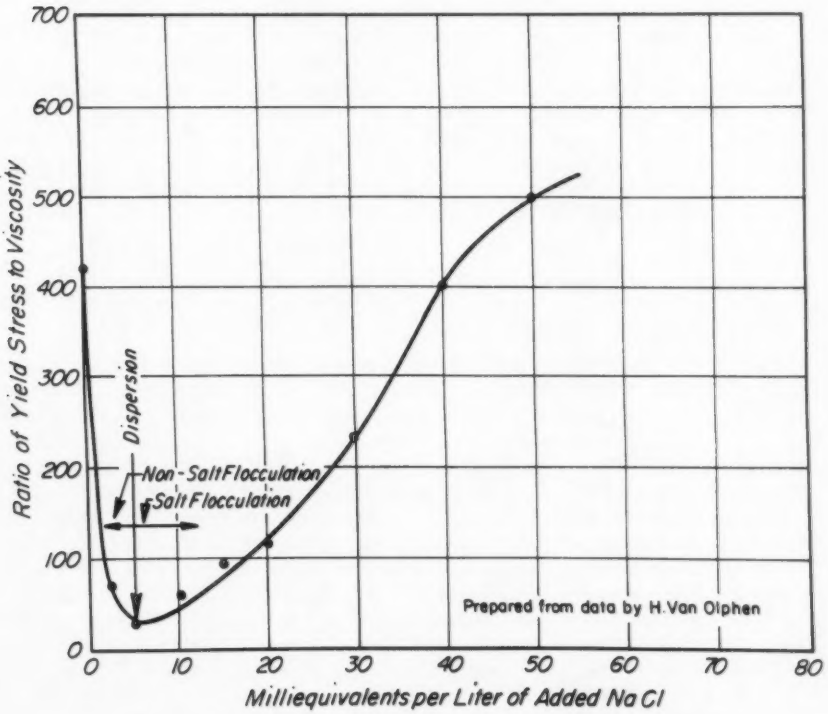


FIG. 11.—EFFECT OF Na Cl CONCENTRATION ON THIXOTROPIC CHARACTERISTICS OF A 3.22% Na-BENTONITE SUSPENSION.

Illustration of the effect of structure on thixotropy in dilute clay suspensions is afforded by data reported by Van Olphen (14). Measurements were made on a 3.22% by weight suspension of pure sodium Wyoming bentonite. Both the yield stress of the gels after a period of rest and the viscosity of the suspensions during shear were measured as a function of added sodium chloride. A measure of the thixotropic effect may be obtained by dividing the gel yield stress by the viscosity. This provides a type of thixotropic strength ratio. Fig. 11 shows the variation in this ratio as a function of added NaCl. It may be seen that with no salt addition the ratio is high, falling to a very low value at 5 m.Eq. NaCl per l and then increasing again rapidly for further addition of salt. Van Olphen was led, by the results of these and other measurements, to the conclusion that at the very low salt concentrations a non-salt flocculation was caused by Coulombic attraction between negative particle surfaces and positive edges. At a salt content of 5m.Eq. NaCl per l the suspension was dispersed and did not gel on standing. With further addition of sodium chloride a salt type flocculation sets in which differs from the non-salt flocculation in that gel times are slower and particles are associated in face-to-face and edge-to-edge associations as well as edge-to-face. (In Van Olphen's view the non-salt flocculated gels are not thixotropic since they set up immediately after stirring. Herein, however, they are considered thixotropic since, according to the definition given in the first part of the paper, no restriction has been put on the time required.)

The importance of this work to the present discussion lies in the fact that only if the suspensions tend to flocculate are they thixotropic. If the suspension remains stably dispersed thixotropic behavior is absent. Thus we have further evidence of thixotropic behavior being caused by a tendency towards flocculation which is over balanced by the dispersing effects of external forces when the material is sheared.

The following quotation from Van Olphen (14) further aids in the understanding of time effects during thixotropic hardening:

> "the thixotropic increase of the modulus of elasticity of a gel with time is a consequence of the increase of the number of links with time after these links have been broken by previous shear. The phenomenon is analogous to slow coagulation."

Effect of Method of Compaction on Thixotropic Behavior.—Since it has been shown by Seed and Chan (23) that method of compaction can have a pronounced influence on the properties of soils compacted wet of optimum, particularly for soils susceptible to a structure change during shear, it would be expected that method of compaction would influence the thixotropic behavior. Samples of Vicksburg silty clay were compacted to the same density at a water content wet of optimum by both static and kneading compaction. Unconsolidated-undrained triaxial tests were performed on these samples at various ages after molding. The stresses required to cause various amounts of strain are shown in Fig. 12 for the two types of compaction. It is immediately evident from this figure that at all ages the static samples are significantly stronger than the kneading samples. This is a logical consequence of the differences in initial structure induced by the two compaction procedures. The differences between the strengths decreases at high value of strain in accordance with the concept of the change of the flocculated structure to a dispersed structure during the test.

The actual strength increase over the 28-day period is slightly greater for the statically compacted samples when measured at low strains and slightly

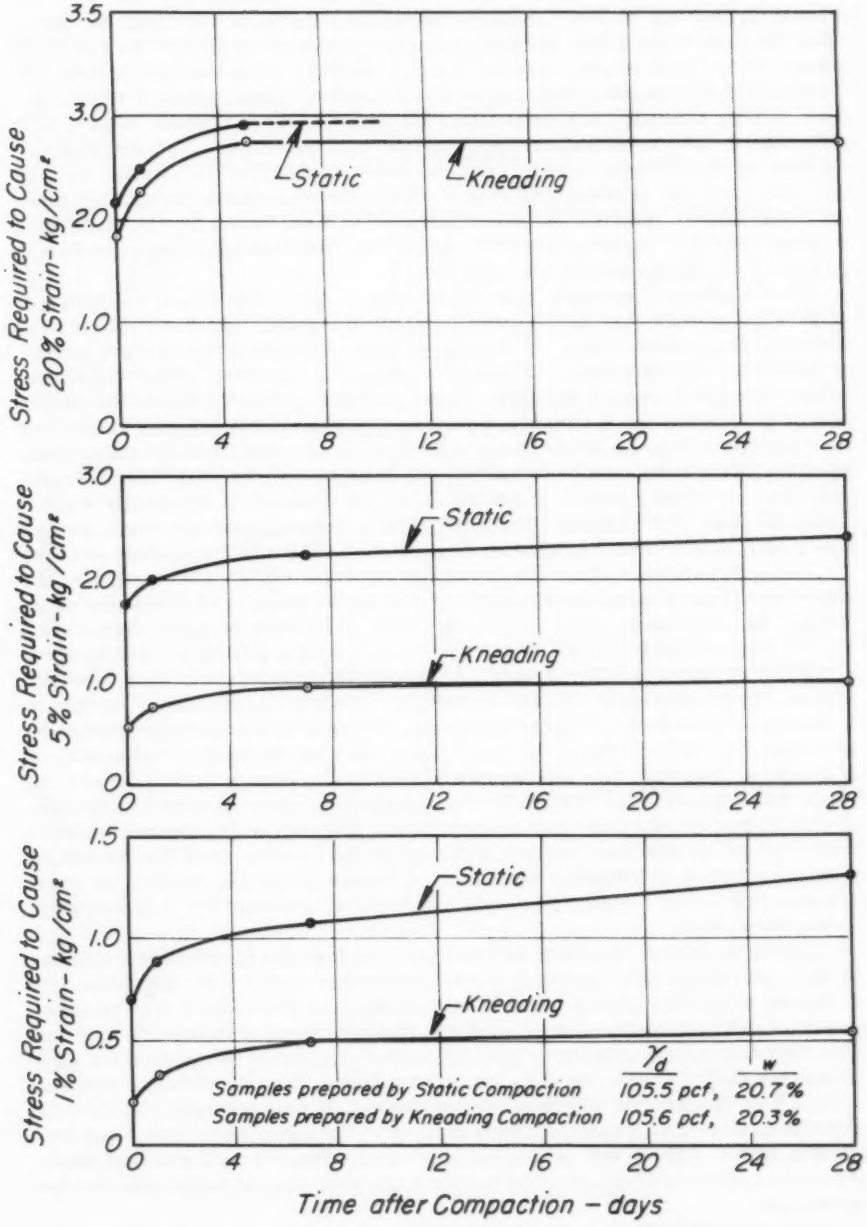


FIG. 12.—EFFECT OF METHOD OF COMPACTION ON THE STRENGTH OF COMPACTED SILTY CLAY.

less when measured at high strains. Quite possibly this greater strength increase in the case of the statically compacted samples is the result of the initial flocculated structure wherein particles are tightly held and very close at so-called "contact points," that is, those points that control interparticle forces. Very slight decreases in spacing in these areas of close approach should result in high strength increases since attraction forces increase rapidly with decreasing spacing at small distances between particles. For the samples compacted by kneading compaction, however, the "effective" initial spacing between particles is somewhat greater due to the more disperse arrangement. At these larger spacings the same decrease in distance would not cause such a great strength increase since the attraction force-distance curve is flatter at larger spacings than for small spacings.

The thixotropic strength ratio is perhaps a more significant indication of thixotropic effects than is the actual strength increase. The ratio reflects the strength change with respect to the initial value and thus offers a more graphic illustration of thixotropic effects for a given initial structure. The thixotropic strength ratio as a function of time is shown in Fig. 13 for the two methods of compaction used. When compared on this basis the samples compacted by kneading are much more thixotropic than those compacted by static compaction. This result might be anticipated because, for the water content used, interparticle forces dictate flocculation in the absence of externally applied shear strains. For samples prepared by static compaction where shear strains are small, a flocculated structure is produced. Since this structure is close to the equilibrium structure for the soil, thixotropic effects are small. On the other hand, the as-compacted structure of samples prepared by kneading compaction is dispersed. As a result the initial structure deviates appreciably from the at-rest equilibrium structure, and thixotropic effects are pronounced.

Effect of Aging on Pore Pressures During Shear.—As noted in a previous section the pressure (or tension) in the pore water of a soil-water system is a measure of the free energy of the water. If free water at atmospheric pressure and 20 C is considered "normal" water then any soil water that registers a pressure less than one atmosphere is at a lower energy level than normal water and any soil water that registers a pressure greater than one atmosphere is at a higher energy level than normal water. Any change in pore water stress during shear or with time reflects a change in the free energy of the system as well as changes in effective stresses. Changes in energy reflect, in turn, changes in particle arrangement, water structure, and electrolyte distribution within the system.

Texts with measured pore pressures were run in order to investigate changes in pore pressure with time and to ascertain whether or not sufficient structural change took place during aging to be reflected by differences in pore water pressures during shear between aged and freshly prepared samples. Seed and Chan (23) have previously shown that dispersed structures develop higher pore pressures during shear than flocculated structures, thus providing a vital and logical link between the effective stress concept of soil strength and the soil structure concept of strength. The structure of the water itself may be an important factor influencing measured pore pressures: a well-ordered water structure leading to lower pressures than a more random arrangement of water molecules.

Typical results of unconsolidated-undrained triaxial compression tests with pore pressure measurements on Vicksburg silty clay are shown in Fig. 14. All points in this figure represent the average of two tests. Four samples

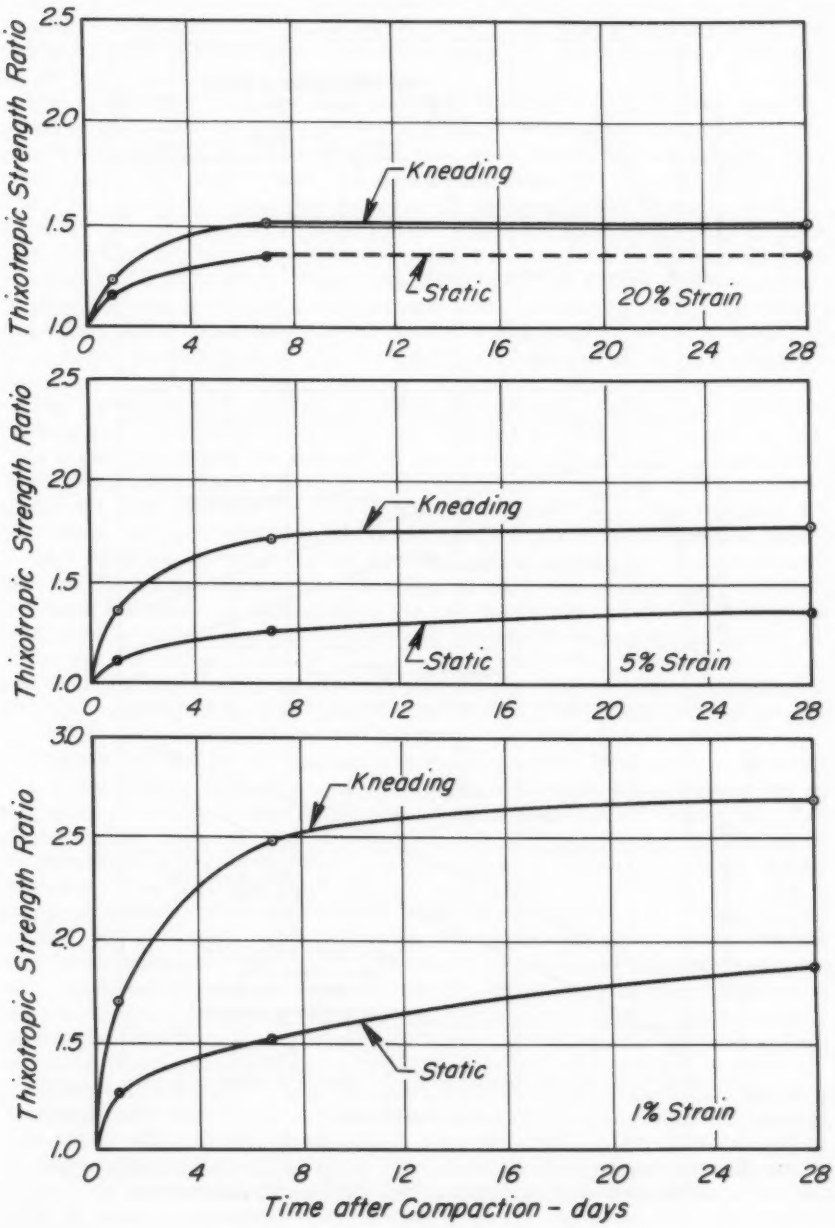


FIG. 13.—EFFECT OF METHOD OF COMPACTION ON THIXOTROPIC CHARACTERISTICS OF COMPACTED SILTY CLAY.

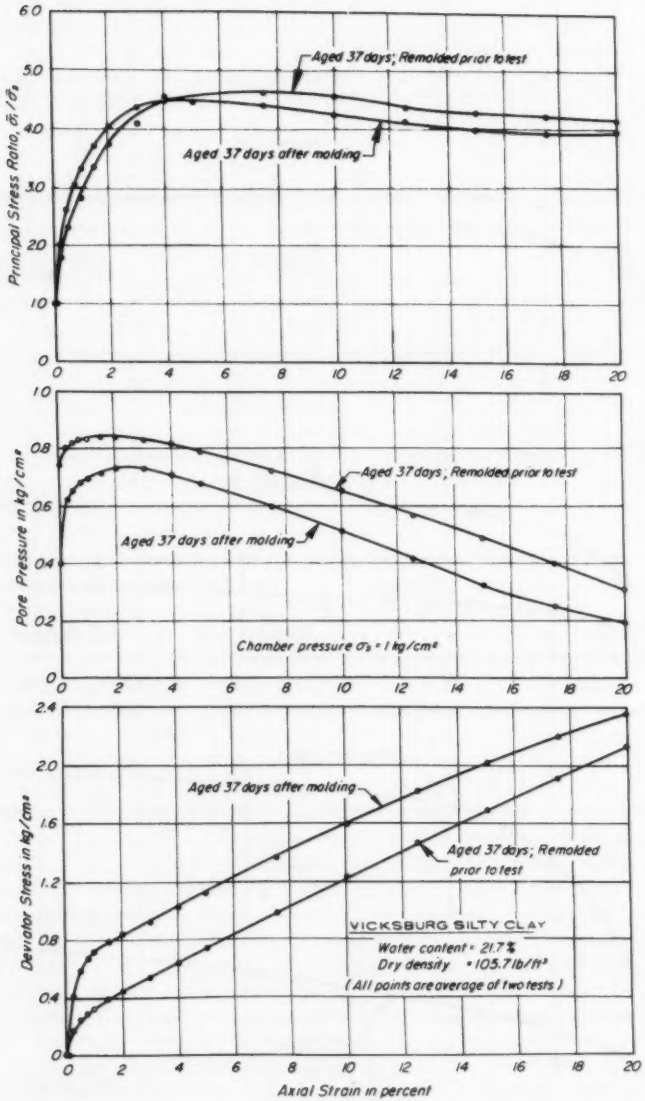

Principal Stress Ratio, σ1/σ3
Aged 37 days; Remolded prior to test
Aged 37 days after molding
Pore Pressure in kg/cm²
Aged 37 days, Remolded prior to test
Aged 37 days after molding
Chamber pressure σ3 = 1 kg/cm²
Deviator Stress in kg/cm²
Aged 37 days after molding
Aged 37 days; Remolded prior to test
VICKSBURG SILTY CLAY
Water content = 21.7 %
Dry density = 105.7 lb/ft³
(All points are average of two tests)
Axial Strain in percent

FIG. 14.—THE EFFECT OF AGING ON THE PORE PRESSURES DEVELOPED DURING SHEAR OF COMPACTED VICKSBURG SILTY CLAY.

were compacted by kneading compaction at a water content of 21.7%, wrapped in rubber membranes, and aged for 37 days. Two samples were then tested without disturbance, while the other two were thoroughly remolded, recompacted to the same density and then tested.

The initial pore pressure value shown on the figure (u at 0% strain) is the pressure that had to be applied to the pore water to prevent either water pick-up by the sample or extrusion of water from the sample. Since the initial values of pore pressures are less than 1.0 kg per sq cm—the chamber pressure—it is evident that the initial pore water in the samples before application of the confining pressure was at a lower pressure than normal water, that is, under tension. (That is, were the chamber pressure zero than the measured pore pressure would be negative. Since pressures are more conveniently measured than tensions, the above described method of testing was found most satisfactory.) The difference between the chamber pressure and the pore water pressure at 0% strain is an indication of both the swelling pressure and the intergranular pressure.

Fig. 14 shows that initially the aged sample has a lower pore pressure (0.39 kg per sq cm) than the freshly recompacted sample (0.74 kg per sq cm). This is consistent with the concept of the development of a more flocculated particle orientation and a more ordered water structure with time in a thixotropic soil. During shear the pore pressures developed in the aged sample remain below those developed in the remolded material. The strength, in terms of total stresses (deviator stress), is significantly greater for the aged sample than for the remolded sample, as would be expected. In terms of effective stresses, however, the strengths of the two materials are about the same as evidenced by the plot of effective principal stress ratio versus strain in Fig. 14. These results are in remarkable accord with data previously reported by Seed and Chan (23) concerning the effect of structure on pore pressure and serve to further substantiate the concept that thixotropic effects result primarily from structure changes with time.

Similar results for compacted San Francisco Bay Mud aged 7 days and tested in the same manner as the silty clay are shown in Fig. 15. Here the initial pore pressure difference between aged and remolded soil amounts to 0.25 kg per sq cm. Again it may be seen that the higher pore pressures develop in the remolded material and that the aged samples are stronger in terms of total stresses. The aged specimens of this soil appear only slightly stronger in terms of effective stresses. A final point worthy of note shown by these tests is that, for both soils, with increasing strain the difference between the two pore pressure curves decreases. The curve for the aged specimens shows a more rapid initial build-up of pore pressure with strain than does the curve for the recompacted specimen. This may be attributed to a rapid breakdown of the thixotropic structure during shear, and is an effect similar to decreasing thixotropic strength ratio with increasing strain as shown in previous figures.

Effect of Thixotropy on Compressibility and Permeability.—If the hypothesis for thixotropic behavior is correct then it must necessarily follow that aging a thixotropic soil will have some effect, however small, on the compressibility, because different structures of the same material consolidated from the same initial void ratio exhibit different compression behavior. Flocculent structures offer a greater resistance to compression under low pressures than do dispersed structures.

Remolded samples of San Francisco Bay mud were studied at a natural water content of 90.4%. Fig 16(a) shows the thixotropic character of this material

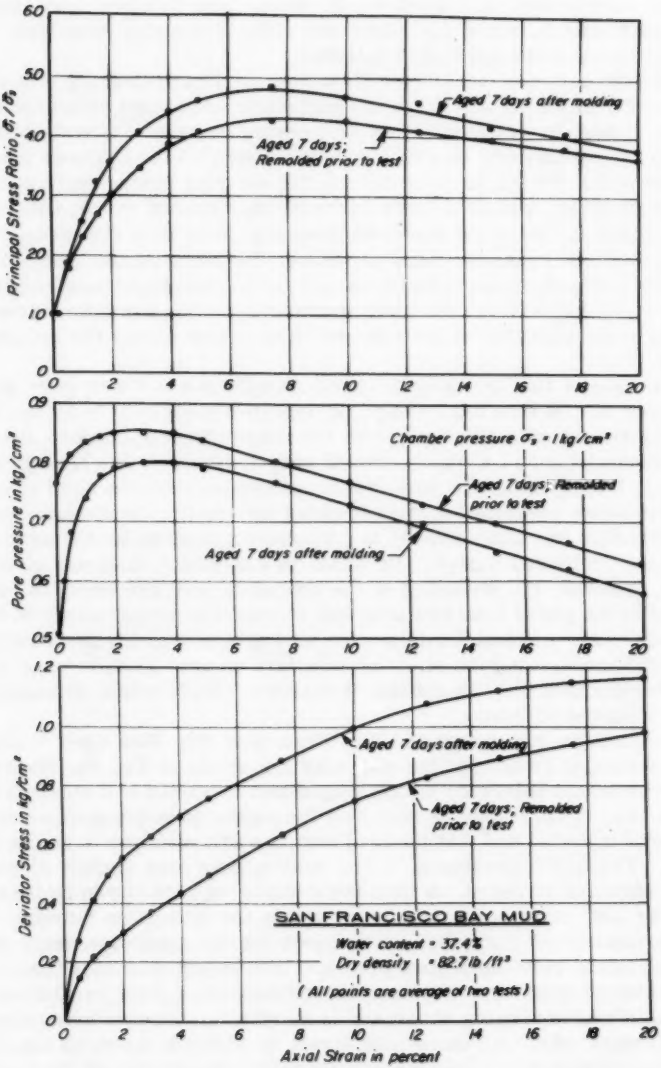

Principal Stress Ratio, σ1/σ3
Aged 7 days after molding
Aged 7 days; Remolded prior to test
Pore pressure in kg/cm²
Chamber pressure σ3 = 1 kg/cm²
Aged 7 days; Remolded prior to test
Aged 7 days after molding
Deviator Stress in kg/cm²
Aged 7 days after molding
Aged 7 days; Remolded prior to test
SAN FRANCISCO BAY MUD
Water content = 37.4%
Dry density = 82.7 lb/ft³
(All points are average of two tests)
Axial Strain in percent

FIG. 15.—THE EFFECT OF AGING ON THE PORE PRESSURES DEVELOPED DURING SHEAR OF COMPACTED SAN FRANCISCO BAY MUD.

as determined by the laboratory vane shear apparatus. Two samples were thoroughly remolded, aged 12 days at constant water content, and tested; two other samples were similarly remolded and aged but again remolded prior to the strength test. Fig. 16(a) indicates significant thixotropic strength gain over a period of 12 days.

Similar samples of this material were tested by means of the standard one-dimensional consolidation test. The void ratio as a function of applied pressure is shown to natural scale in Fig. 16(b) and to log scale in Fig. 16(c). As indicated by Fig. 16 the aged sample has an e-p curve everywhere above that of the remolded sample. That is, aging a thixotropic soil has the effect of in-

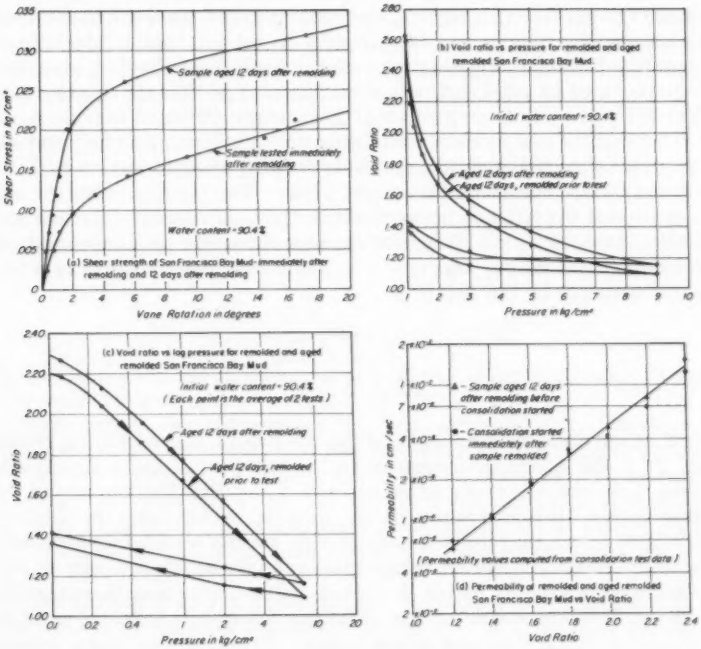

Shear Stress in kg/cm²
Sample aged 12 days after remolding
Sample tested immediately after remolding
Water content = 90.4%
(a) Shear strength of San Francisco Bay Mud- immediately after remolding and 12 days after remolding
Vane Rotation in degrees
(b) Void ratio vs pressure for remolded and aged remolded San Francisco Bay Mud
Initial water content = 90.4%
Aged 12 days after remolding
Aged 12 days, remolded prior to test
Void Ratio
Pressure in kg/cm²
(c) Void ratio vs log pressure for remolded and aged remolded San Francisco Bay Mud
Initial water content = 90.4%
(Each point is the average of 2 tests)
Aged 12 days after remolding
Aged 12 days, remolded prior to test
Void Ratio
Pressure in kg/cm²
Permeability in cm/sec
Δ - Sample aged 12 days after remolding before consolidation started
● - Consolidation started immediately after sample remolded
(Permeability values computed from consolidation test data)
(d) Permeability of remolded and aged remolded San Francisco Bay Mud vs Void Ratio
Void Ratio

FIG. 16.—EFFECT OF AGING ON THE STRENGTH AND CONSOLIDATION CHARACTERISTIC OF SAN FRANCISCO BAY MUD.

creasing the resistance to compression. Examination of Fig. 16 further indicates that above a pressure of 0.1 kg per sq cm the compression curves are very nearly parallel, although a slight tendency towards convergence at some high pressure may be noted. This means that the effect of aging is exhibited under the initial loading; the increased rigidity of the aged sample being responsible for less compression than in the dispersed remolded structure. But once the applied pressure is great enough to overcome this rigidity, the aged sample behaves similarly to the remolded sample. At the same time, because of the time involved in performing a consolidation test, the remolded sample is gaining thixotropic rigidity, which in turn tends to cause similarity in the curves at the higher pressures.

Permeability values were computed from the coefficient of consolidation values as determined by the logarithm of time fitting method measured during each load increment. These values are plotted in Fig. 16(d). Permeability has been plotted as a function of void ratio rather than pressure as it is felt that void ratio is a more rational basis of comparison. Permeability differences between aged and freshly remolded samples at the same void ratio would be directly attributable to structural differences. No significant differences can be noted between the permeabilities of the aged and remolded samples. There is a suggestion, however, that the permeability of the aged sample is slightly higher at the high void ratio (low pressures) where thixotropy has the greatest effect on structure. This would be expected from considerations of the structures of the two systems; however, the uncertainties involved in the computation of permeability from consolidation data do not warrant a detailed analysis of the results. Similar consolidation and permeability behavior was observed in tests performed on aged and remolded samples of Vicksburg silty clay.

Effect of Temperature on the Rate of Thixotropic Strength Increase.—Altering the temperature in a system containing colloids (such as clay) and water is known to affect the status of interparticle forces, the properties of the water and the rates of any reactions that might occur. Thixotropic behavior appears to be a function of the balance between attractive and repulsive forces and further, in thixotropic systems attraction exceeds repulsion by some small amount. The thickness of the double layer, t which controls interparticle repulsion, is given approximately by the relation

$$t = \sqrt[2]{\frac{E\,k\,T}{8\,n\,e^2\,v^2}} \qquad \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

in which E is the dielectric constant of the pore fluid, k denotes the Boltzmann constant, T is the absolute temperature, n represents the concentration of electrolyte, e is the electronic charge, and v denotes the valence of the exchangeable cation. If temperature alone could be varied, then the thickness of the double layer and therefore the repulsive forces would increase with increasing temperature. However temperature cannot be altered without altering the dielectric constant, E, which decreases with increasing temperature. It may be shown that the overall effect of these changes is a decrease in double layer thickness with increasing temperature. Thus, on this basis, increasing temperature causes decreasing repulsive forces between particles. These principles when applied to clays prepared to identical structures, aged at different temperatures, but tested under identical conditions would lead to the expectation that the higher the aging temperature the higher the rate of strength gain. The reason for this is that increasing the aging temperature would accentuate the excess of attractive over repulsive forces, causing a greater driving force for structure alteration to a more flocculated condition than exists in the as-molded state. In addition higher temperatures would lead to decreased water viscosity and increased thermal activity of dissolved ions and the water molecules themselves, which in turn would allow particle reorientations, water structure changes and ionic redistributions to occur more rapidly.

This expectation was confirmed with the following experiments. A mixture consisting of 25% Wyoming bentonite and 75% sand containing sizes between 100 and 200 mesh by weight was prepared to a water content of 24%. Samples were

compacted to a dry density of 100.2 ± 1.0 pcf by kneading compaction. All samples were prepared at a temperature of 70 F, wrapped with two rubber membranes separated by a layer of grease, and stored under water. One-third of the samples were stored at 40 F, a third at 70 F and a third at 140 F. After intervals of storage of 0.29, 1, 3, 7, 14, 24, and 51 days, strength values were measured. All samples were brought to 70 F prior to testing in undrained triaxial compression with a chamber pressure of 1 kg per sq cm. Reliable strength determinations could not be made on the samples aged at 140 F after storage of more than 14 days because the elevated temperature caused deterioration of the membranes followed by adsorption of water by the samples.

Results of these tests are shown in Fig. 17 where the deviator stress required to cause strains of 1%, 2.5%, 5%, and 10% are plotted as a function of time after remolding. The data can be fitted by a straight line on the semi-log plot for times above one day. The rate of strength gain as expressed by the slope of the line for each temperature is as follows:

Temperature in °F	Average rate of strength gain in kg per sq cm per log cycle
40	0.064
70	0.096
140	0.140

It may be seen from the foregoing that the rate of strength gain is more than doubled over the temperature range of 40 F to 140 F.

Strength values were determined on the samples remolded after 51 days storage and are shown in Fig. 17. Due to the water absorption by the samples stored at 140 F meaningful 51-day remolded values could not be determined. However for samples stored at 70 F and 40 F the remolded strength returned almost precisely to the value measured immediately after the initial compaction. This affords additional evidence of the reversible nature of thixotropic effects in soils.

These data are useful in another way. By means of the Arrhenius equation (30) for rate processes the activation energy for the processes involved in the strength changes may be calculated (the activation energy is the energy that any ion, molecule or particle must acquire before it is able to take part in any process, either physical or chemical). The Arrhenius equation is of the form of

$$E_a = \frac{R\, T_1\, T_2}{T_2 - T_1} \ln\left(\frac{k_2}{k_1}\right) \quad \ldots\ldots\ldots\ldots\ldots (2)$$

in which E_a is the activation energy in calories per mole, T_1 and T_2 denote the temperatures in °K at which the rates of property changes are measured, R is the gas constant, and k_1 and k_2 are the rate of change of a given property at temperatures T_1 and T_2, respectively.

If Eq. 2 is applied to the rates of strength increase at 40 F and 70 F an activation energy of 3,900 cal per mole is calculated; whereas, if applied to the

rates at 70 F and 140 F the activation energy is 1,900 cal per mole. According to S. Glasstone, K. Laidler and H. Eyring (30) activation energies of the order of 4,000 cal per mole at about 70 F and 2,800 cal per mole at 212 F are associated with viscous flow of water. This appears a most reasonable phenomenon to be associated with thixotropic effects since particle movements, water redistribution, etc., must necessarily involve movements in the water phase. On the basis of the data at hand no good explanation for the relatively low value of activation energy at the higher temperature is available. It is known, however, that the effects of clay particle surfaces on the properties of the adjacent water phase may be considerable. The effect of the differences in adsorbed wa-

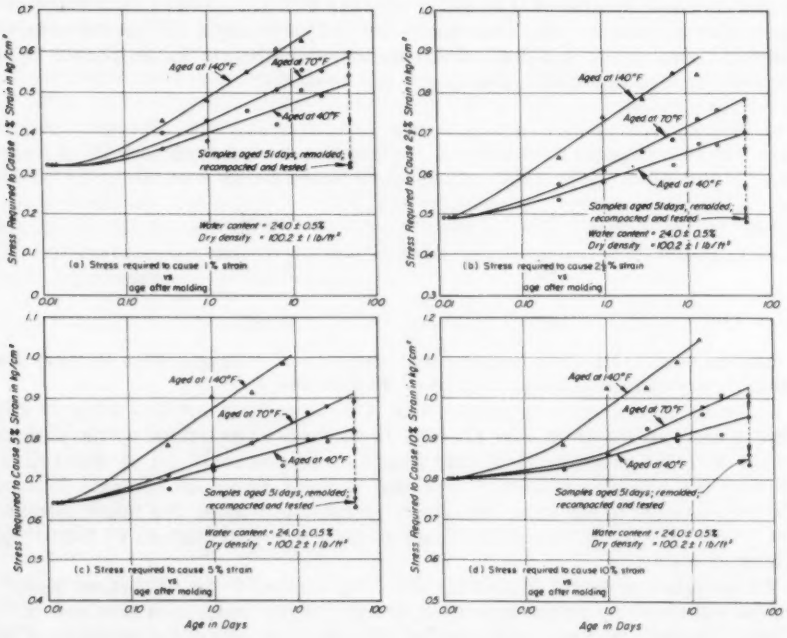

Aged at 140°F
Aged at 70°F
Aged at 40°F
Samples aged 5i days, remolded, recompacted and tested
Water content = 24.0 ± 0.5%
Dry density = 100.2 ± 1 lb/ft³
(a) Stress required to cause 1% strain vs age after molding
(b) Stress required to cause 2½% strain vs age after molding
(c) Stress required to cause 5% strain vs age after molding
(d) Stress required to cause 10% strain vs age after molding
Stress Required to Cause 1% Strain in kg/cm²
Stress Required to Cause 2½% Strain in kg/cm²
Stress Required to Cause 5% Strain in kg/cm²
Stress Required to Cause 10% Strain in kg/cm²
Age in Days

FIG. 17.—EFFECT OF AGING TEMPERATURE ON THE RATE OF THIXOTROPIC STRENGTH INCREASE OF A SAN-BENTONITE MIXTURE.

ter properties from normal water properties on activation energies required for various processes are at this time, unknown.

Although the energy calculations and thermodynamic considerations are somewhat crude when applied to the present data there appear to be great potentialities for the application of such concepts to the study of soil behavior. The feasibility of determining changes in thermodynamic properties associated with such phenomena as creep, secondary compression, and stress-strain behavior has yet to be thoroughly explored; however, should such studies be practical then a much better insight into the fundamentals of soil behavior might result.

PRACTICAL ASPECTS OF THIXOTROPIC BEHAVIOR IN SOIL ENGINEERING

It appears that thixotropic effects can cause significant changes in soil properties, at least over some range of water contents that might be encountered in practice. In compacted soils thixotropic effects may be of importance if the material is compacted on the wet side of optimum. In normally consolidated clays thixotropy may be a contributor to sensitivity, and will most certainly be an important factor when dealing with these materials in the remolded state.

Although theoretical thixotropic strength change equations are not available to permit direct accommodation of thixotropic effects into design problems, certain factors appear worthy of note when considering any problem where a thixotropic soil might be encountered. For example, in any laboratory testing program it would be desirable to place strict controls on the time between sample preparation and testing, in order that all tests be equally affected by thixotropy. In the case of compacted soils, comparisons between the as-molded and soaked properties may be influenced by thixotropic effects occurring during the time required for soaking.

The influence of thixotropy on the Mohr failure envelope based on total stresses for a compacted clay was determined by Seed and Chan (6). The effect of aging was to shift the envelope upwards; the strength intercept was increased, whereas the strength angle remained constant. Thus a total stress design based on the immediate strength envelope for a soil of this type will have an added element of safety due to thixotropic effects. It does not necessarily follow, however, that the strength in terms of effective stresses is increased as a result of thixotropic effects. Reference to Figs. 14 and 15 shows that the principal stress ratio curves for aged samples and samples tested immediately after compaction are not significantly different. Thus, it appears possible that the strength in terms of effective stress is unaffected by thixotropic phenomena.

Reference to the moisture tension data obtained by Day (26) and the initial pore pressure values shown in Figs. 14 and 15 show a very significant decrease in pore water pressure with time in a thixotropic soil. Stated another way, thixotropy causes a marked increase in the pore water tension. The effect of this might be to increase the water adsorption potential of the soil, which could be detrimental in practice.

The effect of thixotropy on compressibility has been indicated in Fig. 16. Two parctical implications of these results may be pointed out. An embankment constructed of a thixotropic soil may show less settlement than anticipated as a result of beneficial structure change with time. Thixotropy, in effect, builds precompression into the soil. Secondly, it is doubtful that a "true" consolidation curve can ever be obtained for a thixotropic soil because of the time involved in testing. Thixotropy and secondary compression work at cross purposes. It would seem reasonable that thixotropic effects during consolidation lead to a smaller compression index then would be obtained if there were no thixotropy.

In the filed when dealing with soft clays, thixotropic effects will probably always be beneficial. The effects of disturbance will decrease with time adding some element of safety to any operation wherein remolding cannot be avoided. An example would be the driving of friction piles into soft clay. The build-up of skin friction with time after driving will be assisted by thixotropic

hardening as well as the dissipation of excess pore pressure in the distrubed zone.

Finally, there seems to be no fundamental reason why thixotropic effects cannot be included in design whenever it appears that they are of sufficient magnitude to warrant consideration, and carefully controlled tests have been run to assess this magnitude.

SUMMARY AND CONCLUSIONS

Previous investigations of thixotropy in suspensions and in soils have been summarized. These studies have pointed out the variety of factors involved in and the complex nature of the phenomena. The importance of such factors as pore fluid composition, particle size and shape and the balance between interparticle attractions and repulsions have all been recognized. Available evidence indicates that thixotropy is of quite general occurrence in fine-grained materials; however a comprehensive theory for the basic cause of thixotropy has not heretofore been available.

An hypothesis based on initial non-equilibrium of interparticle forces after remolding or compaction, and the effects of this non-equilibrium on subsequent structure changes within the soil has been suggested as one possible explanation of the phenomenon. Data consistent with this hypothesis have been presented. It has been found that thixotropic effects will occur if the initial structure is dispersed artificially to an extent greater than dictated by the interparticle forces. Changes in properties occur in response to the structural changes resulting from a dissipation of excess energy from within the system as the soil adjusts its structure to one compatible with the excess of attractive over repulsive forces that exists after the removal of externally applied loads.

Available data indicate that the process is completely reversible, and for the soils tested chemical effects appear minor. The role of water content in determining thixotropic behavior has been found to be primarily one of altering interparticle forces. If it is of appropriate magnitude so that a dispersed structure can be induced by means of externally applied forces even though the soil particles would tend to flocculate when at rest then the soil will be thixotropic. If the water content and double layer conditions are such that the soil assumes a stable dispersed or flocculated structure independently of applied forces then thixotropic effects are negligible.

Pore pressures have been shown to decrease with the aging of a thixotropic soil. This indicates a change in structure of the system which may be attributed to changes in particle orientation, adsorbed water structure and ionic distributions. The decrease in pore pressure reflects the decrease in free energy of the pore water. The pore pressures during shear are lower for an aged thixotropic soil then for one sheared immediately after molding, which is compatible with previous results for flocculated and dispersed structures. Pore pressure measurements have further shown that while thixotropic effects may cause appreciable increases in strength in terms of total stresses they may cause little or no effect in terms of effective stresses.

The effect of thixotropy on one dimensional compression is to raise the pressure-void ratio curve. The greatest effects are exhibited under low pressures. Thixotropy measurements at different aging temperatures have given results in complete accord with the proposed theory, and have further indicated, according to energy calculations that a viscous flow of water may be involved

in thixotropic phenomena. Some of the practical aspects of thixotropy have been pointed out and it is suggested that such effects warrant consideration in design, provided they are accurately evaluated.

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Linearly Variable Load Distribution on a Rectangular Foundation, by A. C. Stamatopoulos. (December, 1959. Prior discussion: None. Discussion closed.)
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Dynamic Testing of Pavement, by W. Heukelom and C. R. Foster. (February, 1960. Prior discussion: None. Discussion closes July 1, 1960.)
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DESIGN AND CONSTRUCTION OF THE AMBUKLAO ROCKFILL DAM[a]

Discussion by Jose O. Lahoz
Addendum to Closure by E. Montford Fucik and Robert F. Edbrooke

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JOSE O. LAHOZ.[1]—The writer, on behalf of the Philippine National Power Corporation, owner of Ambuklao Dam, thanks the authors for the excellent technical presentation on Ambuklao Dam. To further increase the usefulness of Ambuklao in the development of future rockfill dams the writer presents the detailed settlement data for the first 4 yr of service of the dam. The help of J. B. Cooke in preparation of the data is gratefully acknowledged.

The thin central core dam represented the most economical type and the safest with respect to the special foundation conditions. It is 430 ft high and has a crest length of 1,400 ft. The only settlement points are monuments at 50 (164 ft) centers along the crest on the center line of the core, the axis of the dam. Measurements are of vertical movement and up-downstream movement. Cross-canyon movements, that is, lateral movement of the dam from it's abutments were not taken. It is considered that they should be for future large rockfill dams.

Observations and measurements were taken at irregular intervals varying from several days to two months during the first year of operation in 1956. Only one measurement in 1957 is on record. Measurements were resumed in July, 1959 and have continued to date.

Fig. 1 presents the complete vertical and horizontal settlement data for the crest of Ambuklao Dam, together with the related data of reservoir operation and foundation profile. The tabulation presents all the readings for the 4 yr - 0.6 month period of record and also the height of dam at each settlement point, and the 4 yr settlement in terms of percentage of height. The vertical settlement in terms of percentage of height is seen to vary from 0.37% to 1.73% and for the maximum control section at Station 6 + 20 to be 0.77% with a vertical movement of 3.14 ft.

The plotting of "CREST SETTLEMENT - VERTICAL AND HORIZONTAL" profiles the movement of each point. The upstream movement of all points between December 1, 1957 and July 19, 1959 is a measure of the elastic nature of the dam. The water level on December 1, 1957 was 389 ft, and on July 19, 1959 was 232 ft. At Station 6 + 20 the horizontal movement increased 0.29 ft as the reservoir filled between July 19, 1959 and January 8, 1960 while the downward vertical movement was only 0.05 ft. The high vertical settlements at Stations 9 + 48 and 11 + 12, on the steep abutment are of special interest.

The diagram, "RESERVOIR OPERATION", shows the reservoir operation for the full period of service to January, 1960. It is plotted to the same vertical scale and datum as the "VERTICAL PROFILE AT AXIS". The time scale of "RESERVOIR OPERATION" is the same as that for "SETTLEMENT - STA.

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[a] December, 1958, by E. Montford Fucik and Robert F. Edbrooke.
[1] Asst. Genl. Mgr., Philippine Natl. Power Corp., Manila, Philippines.

DATE	TIME	* RES LEVEL	POINT A V	POINT A H	POINT B Vert	POINT B Horiz	POINT C V	POINT C H	POINT D V	POINT D H	POINT E V	POINT E H	POINT F V	POINT F H	POINT G V	POINT G H	POINT H V	POINT H H	POINT I V	POINT I H	DATE
12 22 55	INITIAL	308	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	12-22-55
1 21 56	30 DAYS	311	0	.02	.05	.02	.05	.02	.05	.01	.06	.02	.07	.02	.07	.02	.06	.02	.06	.02	1-21-56
3-7	75 "	318	.05	.05	.05	.05	.07	.05	.08	.07	.10	.11	.11	.12	.12	.14	.13	.11	.13	.08	3-7
5-7	136 "	344	.11	0	.14	.03	.17	.09	.18	.09	.22	.11	.26	.16	.26	.16	.28	.15	.21	.12	5-7
7-13	202 "	349	.12	.01	.17	.07	.21	.12	.24	.09	.34	.16	.41	.23	.45	.27	.46	.26	.41	.17	7-13
9-3	254 "	367	.17	.02	.18	.08	.24	.16	.31	.26	.46	.42	.62	.54	.72	.59	.75	.55	.67	.37	9-3
10-7	288 "	380	.17	.08	.20	.12	.29	.23	.39	.35	.61	.55	.88	.71	.97	.79	1.00	.71	.91	.46	10-7
10-12	293 "	385	.17	.06	.20	.16	.29	.28	.39	.39	.62	.66	.91	.80	1.07	.89	1.12	.81	1.02	.53	10-12
10-15	296 "	387	.17	.03	.20	.15	.31	.28	.43	.43	.69	.70	.97	.88	1.15	.95	1.17	.88	1.11	.56	10-15
10-22	303 "	393	.18	.03	.21	.15	.32	.30	.43	.48	.71	.76	1.04	.96	1.23	1.02	1.27	.96	1.20	.61	10-22
11-11	323 "	394	.18	.04	.23	.17	.39	.38	.53	.69	.88	1.05	1.42	1.30	1.60	1.36	1.67	1.24	1.61	.74	11-11
11-21	333 "	393	.18	.04	.23	.17	.41	.38	.53	.69	.90	1.13	1.55	1.35	1.73	1.39	1.78	1.28	1.72	.79	11-21
12-1-57	1YR 113MO	389	.28	.00	.41	.22	.76	.51	1.02	.88	1.61	1.39	2.48	1.81	2.38	1.88	2.49	1.52	2.34	.90	12-1-57
7-19-59	3 6.9	232	.42	.05	.67	.17	1.12	.47	1.41	.79	2.12	1.35	3.09	1.77	2.86	1.78	3.23	1.47	2.92	.86	7-19-59
9-7	3 - 8.5 "	331	.43	.05	.69	.10	1.14	.43	1.45	.86	2.13	1.38	3.09	1.82	2.88	1.82	3.26	1.48	2.94	.88	9-7
10-10	3 - 9.6 "	357	.43	.05	.69	.12	1.16	.42	1.46	.85	2.14	1.46	3.12	1.85	2.91	1.89	3.31	1.52	3.00	.91	10-10
1-8-60	4 YR 0.6 MO	390	.44	.08	.69	.17	1.16	.48	1.47	.99	2.18	1.56	3.14	2.06	2.91	1.99	3.33	1.62	3.00	.97	1-8-60
HEIGHT AT AXIS FT			.72		1.71		3.12		3.94		4.00		4.10		4.10		3.88		3.74		HT FT
100 x V & H ÷ HEIGHT = %			0.61	—	0.39	0.10	0.37	0.15	0.37	0.25	0.55	0.39	0.77	0.80	0.71	0.49	0.86	0.42	1.73	0.56	% HT
			0+00		1+64		2+28		3+92		4+56		6+20		7+84		9+48		11+12		12+76

* RESERVOIR LEVEL IN FEET DATUM IS LOW POINT OF END PROFILE AT AXIS. FULL RES. EL. 396, CREST 409.7

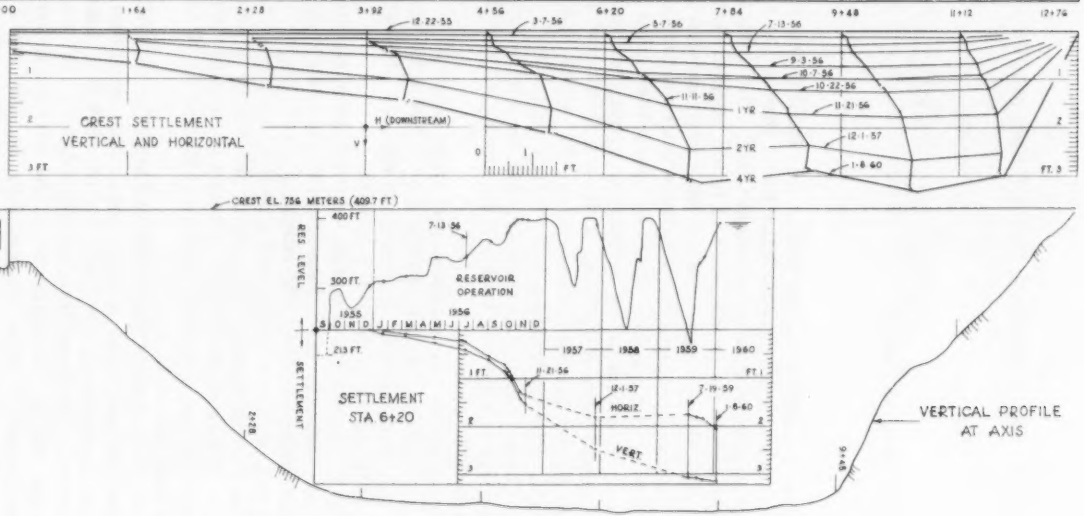

FIG. 1.—CREST SETTLEMENT, PROFILE AT AXIS, AND RESERVOIR OPERATION AMBUKLAO DAM

6 + 20" in order to show the relation between reservoir operation, time and settlement for the maximum section STA. 6 + 20. Any movement that might have occurred between completion of the fill in June, 1955 and December 22, 1955, the date of initial permanent monument reading is not recorded. The maximum rate of settlement was in October and November, 1956 as the top 30 ft of the reservoir was filled for the first time. The settlement during the long intervals of time from November 21, 1956 to December 1, 1957, and from December 1, 1957 to July 19, 1959 are shown dotted since the shape of the curve is not known.

At the beginning of this discussion the writer mentioned, "special foundation conditions". The paper covers the geology of the site, but some additional comment may emphasize the consideration given to adapting the dam to the site. In its 1951 "Report on the Development of Power in the Agno River in Luzon, at the Ambuklao Site", the National Power Corporation, owner of the Ambuklao Dam, states in part that ". . . by numerous borings and test pits, it is known definitely that no fault crosses the dam site under the river; that the depth to bed rock under the river is moderate, averaging about 7 meters; and that the condition of the rock under the river is good. The left abutment ridge is composed of diorite and metamorphic rock with very complex relations. One of the fault zones underlies the gully on the upstream side of the ridge, and the other intersects the far end of the ridge but does not extend to the river downstream from the ridge and cannot form a leakage path thru the ridge. The fault zones do not constitute a serious weakness for the type of dam planned. The impervious earth core will cover the fault zones except at the extreme upstream toe of the dam where they will be covered by the rockfill."

E. MONTFORD FUCIK,[2] F. ASCE and ROBERT F. EDBROOKE,[3] M. ASCE.—Mr. Lahoz has presented a valuable record of the horizontal and vertical movements of the crest of Ambuklao Dam. This record, which was not previously available to the writers, shows the maximum vertical settlement to be 3.33 ft at Sta. 9 + 48. This is considerably more settlement than that cited by the writers, but is well within the expected range for a vertical core rockfill dam. The crest of Ambuklao Dam was "overbuilt" about 3.25 ft to provide for future settlement, so it appears that the maximum settlement has exceeded this by about 1 in. In general, however, the percentage vertical settlement is of the same order of magnitude as that experienced at other central core rockfill dams.

The horizontal movements were considerably less than the vertical movements. The largest horizontal movement was 2.06 ft downstream at Sta. 6 + 20 midway between the abutments. At least half of the total horizontal movement recorded in the 4-yr period of record took place during the initial raising of the reservoir. There is some indication that horizontal rebound occurred when the reservoir level was lowered.

The writers agree with Mr. Lahoz that measurements of lateral movements could provide useful information, and such measurements should be taken on future vertical core rockfill dams.

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COMPRESSIBILITY AS THE BASIS FOR SOIL BEARING VALUE[a]

Discussion by John A. Focht

JOHN A. FOCHT, JR.,[1] M. ASCE.—Mr. Hough points out, towards the end of his paper, that there exists tacit assumption on the part of many designers that movement analysis in footing design is not required if the factor of safety against rupture is adequate. A careful reading of any of the standard texts on soil mechanics[2,3,4,5] would not lead a reader to this assumption, which certainly is not universally correct. Even less comprehensive texts[6,7] discuss the problem of designing footings to minimize differential settlement. To reach the above-mentioned assumption, which the writer agrees is prevalent to some degree, there must be mis-interpretation, partial reading or misunderstanding on the part of a user of a standard text. Therefore, it seems that Mr. Hough's argument as stated in the first sentence of his abstract is directed largely at the engineer who might be termed a "careless" reader, rather than at the usually quoted authorities.

By including Fig. 3 in what is otherwise a generally logical paper, the author has invited misinterpretation and misapplication of his paper. Furthermore, there are at least three significant assumptions made in his development which impose severe limits on the validity of results obtained; but there is little or no mention of these limitations. These four "errors" in the paper could easily lead the "careless" reader to another tacit assumption—"footings can be designed following the simple procedures outlined, using Fig. 3, and designing for the sustained structural load,—with the result that there will be no danger of soil rupture or of differential settlement." This assumption is as wrong as the one Mr. Hough is attempting to combat and careful reading of his paper clearly shows that he would disagree with such a broad assumption derived from it. Nevertheless, the writer believes that more readers will be misdirected than will be corrected unless the severe limitations of Fig. 3 are emphasized and a few other limitations are pointed out.

The correlations which have been suggested by others between standard penetration resistance, N, and the relative density of sand are poor at best.

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[a] August, 1959, by B. K. Hough.

[1] Chief Design Engineer, McClelland Engineers, Inc., Houston, Texas.

[2] "Soil Mechanics in Engineering Practice," by K. Terzaghi and R. B. Peck, John Wiley and Sons, 1948, pp. 411–412.

[3] "Fundamentals of Soil Mechanics," by D. W. Taylor, John Wiley & Sons, 1948, pp. 605–607.

[4] "Soil Mechanics, Foundations, and Earth Structures," by G. Tschebotarioff, McGraw-Hill Book Co., 1952, p. 371.

[5] "Basic Soils Engineering," by B. K. Hough, The Ronald Press Co., 1957, p. 277.

[6] "Foundation Engineering," by R. B. Peck, T. Thornburn, and W. E. Hansen, John Wiley & Sons, 1953, pp. 216–217.

[7] "Substructure Analysis & Design," by P. Andersen, The Ronald Press Co., 1948, pp. 78–85.

The use of N to evaluate the shear strength of clay is appropriate only if a very rough estimate of strength is desired. Such correlations would probably not be used for close design of Mr. Hough or any one else familiar with the very broad scatter of points through which the correlating line has been drawn. For years it has been the writer's opinion that the various "rule-of-thumb" correlations developed using N have hindered the advance of soil mechanics rather than helped because they tend to lead to "handbook" designs. Therefore, it is considered extremely unfortunate that an authority such as Mr. Hough introduces in Fig. 3 a correlation of compressibility and N without any substantiating data whatsoever, even if he wanted only to show the order of magnitude of the bearing capacity index.

The general validity of the simplified approach for footing design suggested by Mr. Hough hinges on several limiting assumptions either not mentioned at all or touched on only briefly; some of these are—

1. The sustained footing load is equal to or is a large percentage of the maximum footing load.
2. The soil conditions are uniform in both horizontal and vertical directions.
3. The soil compressibility, as expressed by C, does not vary with depth or with the magnitude of the applied load.

Deviation of actual conditions from these assumptions does not in any way detract from the value of the theoretical development of Eqs. 7 through 37 and Figs. 5 through 8 in pointing out the relative importance of the dimensional factors involved in footing settlement. Nor is the appropriateness of Mr. Hough's encouragement of more attention to movement analysis affected. The inexactness of these assumptions is not, however, consistent with the implication of Fig. 5 and other parts of the paper that movements can be computed by the simplified procedure with a precision of 0.01 ft to 0.02 ft.

There are a variety of structures in which the sustained load, or the normal operating load, is a small fraction of the maximum design load. Transmission towers and tall process columns are in this category. Designing footings of such units on the basis of settlement under normal load might not provide any margin of safety for rupture under maximum load. Mr. Hough's paper seems to be directed primarily at design of footings for residential, commercial, and light manufacturing buildings where the footing loads are primarily dead loads and the loads are all in the same order of magnitude. As a side issue, no consideration was given to those structures which are not movement-sensitive. In any case, design of footings can not be divorced from the characteristics and requirements of the structure to be supported.

If soil conditions at a site are generally uniform, a simplified approach for equalizing footing settlement using precomputed charts and graphs like Fig. 5 may be satisfactory. However, in the writer's experience, soil conditions are seldom completely uniform over a large site; and proper use of the simplified method, as any other, will require evaluation of the varying conditions and the varying parameters of soil characteristics. A designer who has been consistently averaging N values and using a family of charts like Fig. 5 probably has also lost sight of the basic principles involved in eliminating differential settlement. As a consequence he will be lead by this paper to follow the suggested simple "handbook" procedure using incorrectly an average parameter for compressibility, even tho the parameter actually varies over the site. The writer feels that where conditions vary, the designer who approaches footing design relying on basic fundamentals will produce better designs because of his better

understanding of the problem than the designer who uses a simplified, chart-type, handbook approach as developed in this paper.

Even in a uniform soil deposit, compressibility may vary with depth. In sands it is commonly accepted that the compressibility (used as a broad term) may decrease with depth. Where the foundation medium is a stiff overconsolidated clay, the degree of overconsolidation often decreases with depth which should mean a variation in compressibility with depth. In a uniform clay deposit, the compressibility decreases with depth because of the decreasing ratio of additional stress to original stress. In addition to these variations, a consolidation test curve is a curve rather than a straight line for most soils except the normally consolidated clays; therefore, the compression index, C_c, is often a secant approximation of the curve and is itself a function of the stress increment. Using the example on Fig. 5 with C = 100 and considering footing sizes ranging from 4 ft to 10 ft, the allowable contact pressure, p'_c, will range from 3,600 psf to 6,200 psf, and according to Fig. 6 the depth of effective stress will range from 10 ft to 15 ft. It would be only the most uniform soil condition which will not vary in compressibility with nearly 100% variation in applied pressure and a 50% variation in the depth of soil participating in footing movement.

Because of the various factors mentioned above but which were not adequately discussed in the paper, the assumption of uniform and constant compressibility is valid only for very limited ranges of footing sizes and bearing pressures, and even then only for an obviously uniform soil. This fact coupled with consideration of normal variations in soil properties between borings indicates that direct application of Eqs. 7 through 37 and a set of charts like Fig. 5, should be done with extreme care; otherwise, as the author concluded for the situation of a careful site investigation and compliance with code regulations, " -- chances of differential settlement --- may well remain."

Despite the variance of the above comments with the paper, the writer and Mr. Hough do agree on the two fundamental criteria of foundation design:

1. Each individual foundation must safely transmit its maximum design load to the soil without danger of a shear failure in the soil.
2. The total and differential foundation movements (usually computed for average sustained load) must not impair or affect the utility of the supported structure.

They can be applied in either order, but in every case both criteria must be given some degree of consideration, even if only to decide that one is not of consequence for the particular problem. It will make little difference to the completed structure, when its footings are properly designed, which criterion was used first in the design. It is the responsibility of the foundation engineer to evaluate all of the factors involved and then to select a design procedure which will yield a satisfactory foundation design for the contemplated structure.

SUBSURFACE EXPLORATIONS IN PERMAFROST AREAS[a]

Discussion by G. H. Johnston, G. Robert Lange, Henry W. Stevens and William P. Verville

G. H. JOHNSTON.[1]—The need for adequate subsurface investigations at proposed construction sites underlaid by perennially frozen ground cannot be overemphasized. The methods and techniques used to obtain the required site information differ somewhat from those generally followed in more southerly regions. Unfortunately, very few records of subsurface explorations in permafrost areas have been published. It was a pleasure, therefore, to read the account of the work at Frobisher Bay, Baffin Island, Northwest Territories, Canada, by Mr. Cass.

The author indicates that considerable difficulty was experienced in obtaining samples of frozen material when using a churn drill and drive sampling methods. This was particularly true in the coarse-grained soils of which few samples were procured. The author further reports that much better results were obtained (although still rather incomplete and erratic) by the use of a core drill, and on this basis, he concludes that "subsurface explorations in the Arctic are best accomplished by core drilling methods."

Canadian experience has confirmed the difficulties in using a churn drill and drive sampling methods to obtain suitable samples of frozen soils, and has indicated this technique to be generally unsuitable for exploration in permafrost areas because of the difficulty of penetrating the frozen ground in coarse-grained soils and of extracting material from the sampler in fine-grained soils. In the case of core drilling, a number of detailed investigations[2,3,4,5] have been carried out in recent years to develop and improve drilling methods and techniques for obtaining frozen core samples. Although these studies have shown that core drilling can be successfully carried out in all types of frozen material, the special equipment, materials, and techniques required, particularly for coarse-grained soils, are such that this method may not always be practicable. For instance, its use may be limited by difficult access conditions,

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[a] October, 1959, by James R. Cass, Jr.

[1] Northern Bldg. Sect., Div. of Bldg. Research, Natl. Research Coun., Ottawa 2, Canada.

[2] "Permafrost Investigations at Aklavik: 1953 (Drilling and Sampling)," by J. A. Pihlainen and G. H. Johnston, Div. of Bldg. Research, Natl. Research Council, Canada, (NRC 3393), January, 1954.

[3] "Core Drilling in Frozen Ground," by M. J. Hvorslev and T. B. Goode, Seventh Annual Drilling Symposium, Exploration Drilling, Univ. of Minnesota, Center for Continuation Study of the General Extension Div., Minneapolis, October, 1957, p. 114.

[4] "Interim Report on Core Drilling in Frozen Ground," by W. K. Boyd and G. R. Lange, Seventh Annual Drilling Symposium, Exploration Drilling, Univ. of Minnesota, Centre for Continuation Study of the General Extension Div., Minneapolis, October, 1957.

[5] "Refrigeration of Fluids for Drilling and Coring in Frozen Earth Materials," by G. R. Lange, Paper presented to the First Internatl. Symposium on Arctic Geol., held in Calgary, Alta., January, 1960.

cost, weight, and quantity of the equipment required, difficulty of movement of this equipment about a site, scope of the investigations to be carried out and climatic conditions.

An alternative method in such cases, and one that is worthy of careful consideration is the use of test pits. Test pits have particular application to site exploration in northern Canada where large areas are covered with deposits of very stony glacial tills and coarse gravels. These materials can contain large amounts of ice in the form of thin layers or lenses, coatings on individual particles, or inclusions several feet thick; and it is extremely important that these soils be very carefully examined prior to construction.

An important advantage of test pits is that they permit the frozen soil and the ice segregation to be examined in detail in the undisturbed condition. Test pits can be excavated fairly cheaply and rapidly; they require a minimum of special equipment, and do not require skilled labor, thus, avoiding the need to bring men in from the outside when local labor is available. In addition, investigation of several areas can be undertaken at one time.

Test pits can be advanced in several ways—by solar thawing, artificial thawing (steam, fires, and so forth) pick and shovel, air, or gasoline powered jackhammers, explosives, or by mechanical excavating equipment. By far the most effective method of excavation, however, appears to be excavation by the use of jackhammers powered by compressed-air or gasoline engine. Test pits are best excavated in the early spring months when the ground is completely frozen, although many have been dug successfully during the summer period. Some cribbing for the first few feet is generally required, if excavation is carried out in the summer months. However, the remainder of the excavation is usually sufficiently stable to stand without cribbing because of the frozen condition of the material. Water entering the excavation due to thawing of the ground, from surface seepage, or when unfrozen layers (taliks' are encountered, can generally be handled by bailing or the use of a small pump.

Normally, foundation investigations are required to depths of 20 ft to 30 ft. Two or three laborers, who alternate working in the excavation, can advance a test pit at the rate of 10 ft per 10 hr day with most pits being dug to a depth of at least 20 ft. Undisturbed samples can be taken where desired from the walls of the pit and the soil profile logged at intervals as the work proceeds. Thus, no information is missed and the rate of progress compares favorably with that obtained by core drilling methods. An all-inclusive cost of a test pit operation has been estimated at approximately $7 to $10 per ft.

In contrast, the costs of core drilling are generally higher but not as great as $44 and $42 per ft as reported in the paper by Mr. Cass. Similar investigations in the western Arctic, along the Mackenzie River and at Frobisher Bay, using medium-sized diamond core drills, have been carried out at a cost of no greater than $20 per ft and, in some cases, at a cost of about $7 per ft. Many variables make cost comparisons difficult but even when the location of the work and the high cost of transportation are taken into account and, also, realizing that part of the work consisted of attempts to improve the sampling techniques, the costs quoted appear abnormal. The value of planning an operation well in advance cannot be underestimated and does much to decrease the overall cost. Generally, a program must be planned from six months to one year ahead of the actual operation in order to take advantage of the shipping season, site facilities and the periods of the year when the work is best carried out.

The author mentioned that the Snow, Ice and Permafrost Research Establishment of the United States Corps of Engineers, Dept. of the Army, method

for describing and classifying the ice segregation in frozen soils was used. The use of such a system to establish uniformity in reporting the ice phase in frozen soils is highly recommended. The Snow, Ice and Permafrost Research Establishment system has been used by the Division of Building Research of the National Research Council of Canada for several years in all permafrost investigations, and has been extremely useful. Some modifications to this system have been made as a result of experience gained in its use chiefly with regard to differentiating between thin ice lenses (up to 1 in. thick) and "massive" ice inclusions (over 1 in. thick) but, essentially, the method of reporting is the same. The value of this paper would have been considerably enhanced had several of the borehole logs been reproduced giving in more detail the types of soils encountered and, particularly, the type and distribution of ice in these soils.

Published records of subsurface investigations in permafrost areas such as those reported by Mr. Cass can be of immediate value to those concerned with this phase of northern construction and development. It is hoped that others will follow his example in reporting their experiences for, in recent years, there have been a number of investigations worthy of note carried out in the North.

G. ROBERT LANGE.[6]—Mr. Cass has described some of the problems of foundation exploration in permafrost in a manner that will be valuable to anyone contemplating a similar job. His description of these problems points out the need for improved methods and equipment designed especially for the exploration of rock permafrost and procurement of undisturbed samples of course-grained as well as fine-grained frozen soils.

The United States Corps of Engineers, Dept. of the Army's Snow, Ice and Permafrost Research Establishment implemented a research project several years ago which, it was hoped, might furnish at least a partial solution to some of these problems. It was recognized that a positive method of maintaining the hole wall and core in the frozen state, during high ambient air temperatures, would be required. A portable refrigeration system that would cool an adequate flow of diesel fuel or compressed air (as a rotary drilling fluid) to +15°F at ambient air temperatures of up to 70°F evolved from the work conducted by this organization. It has been used with considerable success in various frozen soils, including frozen river gravels.

HENRY W. STEVENS[7] and WILLIAM P. VERVILLE.[8]—Mr. Cass has performed a needed service in describing his experiences in sampling frozen soils in permafrost areas. Pooling the knowledge gained by such experiences can help to establish reliable methods.

The investigations described by Mr. Cass confirm that drive sampling is satisfactory in frozen, fine-grained soils at comparatively warm soil temperatures. No data were given on the latter factor but it may be inferred from the time of year and the location. It is probable that the warm soil temperatures were an important reason why rotary core drilling, with brine solutions at only

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a degree or two below freezing temperature, was unsuccessful. The successful use of compressed air, cooled by exposing to cold air temperatures, augers well for the success of the current investigations[9] of the Snow, Ice and Permafrost Research Establishment, U. S. Army, Corps of Engrs., Wilmette, Ill. (USA SIPRE) into the use of refrigerated compressed air.

The Arctic Construction and Frost Effects Lab., U. S. Army Engineer Div., New England, Waltham, Mass., (ACFEL) has had considerable experience with exploration in frozen ground by drive sampling methods. Many situations encountered, particularly in Alaska, were such that drive sampling was most feasible and economical. Indeed, in some cases, drive sampling had decided advantages, not the least of which was that it could be accomplished without the use of water, cooled fluids or other auxiliary equipment. However, it is apparent that drive sampling is only possible in the finer-grained soils at comparatively warm soil temperatures. Exactly what conditions are necessary for success have not been defined, at least in print, and the average engineer may be somewhat uncertain whether to depend upon drive sampling or resort to the

TABLE 1

Soil Type	Water Content % by Weight	Sample Diameter	Approximate Soil Temperature
Brown clean uniform SAND (SP). No segregated ice.	25.0	2 in. I. D.	29°F
Light grey, poorly graded, medium to fine SAND (SP). No ice segregation.	18.0	2-3/4 in. I. D.	30°F
Sandy, clayey SILT (CL-ML), Segregated ice	38.7	1-1/2 in. I. D.	27°F
Grey SILT (ML), Segregated ice, organic contents 3.8%	86.7	2-3/4 in. I. D.	30°F
Grey SILT (ML), Segregated ice, organic content 4.3%	129.4	2-3/4 in. I. D.	30°F

more complex and expensive rotary drilling. ACFEL is presently compiling statistics from past experience in an attempt to define the uses of drive sampling in frozen soils as well as suggesting equipment to accomplish a given job.

A few frozen soil types that have been successfully drive sampled are here listed in Table 1 and may serve to point out possibilities not realized.

It is very important, of course, that rotary drilling methods in frozen soils be developed to their optimum usefulness because of the many situations in which drive sampling is impossible or will not produce the desired information. Recent reports of experiences in rotary drilling have provided much "know how." ACFEL enjoyed the opportunity of working with U. S. Army Waterways Experiment Station, Vicksburg, Miss., in rotary drilling coarse glacial tills in Greenland, in 1956-1957, in which excellent results were obtained at relatively low subsurface temperatures.[10] SIPRE's very promising investigation into the

[9] "Interim Report on Core Drilling in Frozen Ground," by W. K. Boyd and G. R. Lange, Seventh Annual Drilling Symposium, Univ. of Minnesota, Minneapolis, Minn., 1957.

[10] "Core Drilling in Frozen Ground," by Dr. M. Juul Hvorsley and Thomas B. Goode, Seventh Annual Drilling Symposium, Univ. of Minnesota, Minneapolis, Minn., 1957.

use of refrigerated compressed air and rotary drilling has already been mentioned. However, there is still need for development in methods and equipment to provide reliable procedures for all situations and locations. A particular need is for lightweight portable equipment that can be transported in relatively small planes to remote areas where water or other supplies are unavailable. At the same time, the equipment should be capable of sampling any type of frozen soil to depths of at least 100 ft. It is hoped that all who have experience and can contribute to the general knowledge of this important matter, will do so.

RATE OF CONSTRUCTING EMBANKMENTS ON SOFT FOUNDATION SOILS[a]

Discussion by Harry R. Cedergren, F. P. Silva, D. H. Gilbert, Jr.

HARRY R. CEDERGREN,[1] M. ASCE.—The author points out that "There has been a great deal of misunderstanding and misconception in the past regarding the use of apparent and effective angles of internal friction in estimating shear strength." Although a logical explanation of wide differences in shear coefficients that may be obtained on saturated clays was offered by Arthur Casagrande, in 1934,[2] there still is a great deal of misunderstanding and misconception regarding the use of angles of internal friction of clays. For example, in engineering publications one can find references to friction angles of 0° to 15° for San Francisco bay mud, however, careful analysis of a substantial number of well-performed triaxial shear tests on "bay mud" consistently indicate a true angle of internal friction of approximately 30°. Soft, normally consolidated mud or clays of this kind stubbornly resist increases in strength. Their low permeability is responsible for the extremely slow flow of water out of the soil pores, thus, the low permeability is responsible for a very slow increase in strength. When shear coefficients are computed from the total stresses applied to laboratory specimens of clay that are not fully consolidated, low "apparent" friction coefficients are obtained. If allowance is made for pressures in the pore water, much higher friction coefficients are indicated. This point is substantiated by the triaxial shear data in Table 1. In column 1 of Table 1 is given the consolidation pressure (the chamber pressure, or "all-around" pressure) under which the specimens were consolidated. In column 2 is given the chamber pressure at which the shear test was performed, during the application of which no further drainage was permitted. The deviator stress in column 3, may be noted to be basically independent of the chamber pressures in column 2. All "circles of stress" that would be obtained by plotting Mohr diagrams with the total externally applied stresses (column 2 as "minor principal stress" and the sums of columns 2 and 3 as "major principal stress") give low "apparent" coefficients of friction. Since the deviator stresses in column 3 show no measurable increase with pressures applied after consolidation, it can be concluded that for these tests essentially 100% of all stress applied after stopping drainage is transmitted to the pore water. The effective axial stress in each series of 3 tests is, therefore, essentially equal to the consolidation pressure listed in column 1, and the effective lateral pressure basically equal to the consolidation pressure minus the deviator stress in column 3. These values are listed under columns 4 and 5 as very close approximations of the "effective" stresses at failure. A Mohr diagram plotted

[a] October, 1959, by Herbert L. Lobdell.

[1] Senior Materials and Research Engr., Calif. Div. of Highways, Sacramento, Calif.

[2] Journal of the Boston Society of Civ. Engrs., July, 1934, (Reprinted in "Contributions to Soil Mechanics - 1925 - 1940," BSCE, 1940, pp. 218-225.

for these tests using the "effective" stresses thus calculated gives a friction coefficient of approximately 30°. When the total externally applied pressures are plotted on Mohr diagrams, "apparent" friction coefficients of 0° to 15° can be obtained.

The test data discussed above were obtained on non-organic soft silty clay from the San Francisco Bay area. Similar data are available from widely scattered locations in that area. These tests would seem to place credence on the concept that the strengths of at least some clays can be attributed primarily to friction. Certainly, it is evident that any stability studies relating to soft clays should be based upon a clear understanding of the manner in which strength coefficients are obtained. This particular group of tests was made on 2 in. dia. specimens, consolidated several weeks essentially to equilibrium and then sheared in 10 or 15 min without further drainage.

When the writer first applied the consolidation theory to a stability analysis of an earth dam on a clay foundation (in 1940) he met objections from some

TABLE 1.—TRIAXIAL SHEAR DATA[b]

Consolidation Pressure (1)	Chamber Pressure at Shear (2)	Deviator Stress (3)	Effective[a] Axial Pressure (4)	Effective[a] Lateral Pressure (5)
0.5	0.5	0.38	0.5	0.12
0.5	1.0	0.36	0.5	0.14
0.5	2.0	0.40	0.5	0.10
1.0	1.0	0.73	1.0	0.27
1.0	2.0	0.74	1.0	0.26
1.0	3.0	0.70	1.0	0.30
2.0	2.0	1.33	2.0	0.67
2.0	3.0	1.26	2.0	0.74
2.0	4.0	1.32	2.0	0.68

[a] Assuming that after consolidation is stopped all added stresses are transferred to the water.
[b] All values in tons per square feet

practical engineers on the grounds that the method was just too theoretical. Observation of long-time records of structures on soft mud foundations have convinced the writer that these methods, if used intelligently and with respect for their limitations, can be of substantial value in predicting the behavior of full-size projects on clay foundations.

In carrying out an analysis of the safe rate of constructing embankments on soft foundations, the writer usually calculates the average per cent consolidation of the foundation required to support the fill at progressively greater heights. One does not know in advance the most suitable thickness of fill increments at various stages of construction. To avoid repetition of the stability analysis for design and during construction one can select a number of fill heights covering the required range, and calculate from a stability analysis the required per cent consolidation for each height. Thus, for example, if the gross height of a fill is 25 ft, stability analyses might be made at heights of 10 ft, 15 ft, 20 ft and 25 ft. These heights do not need to have any relation to the thickness of lifts applied during construction.

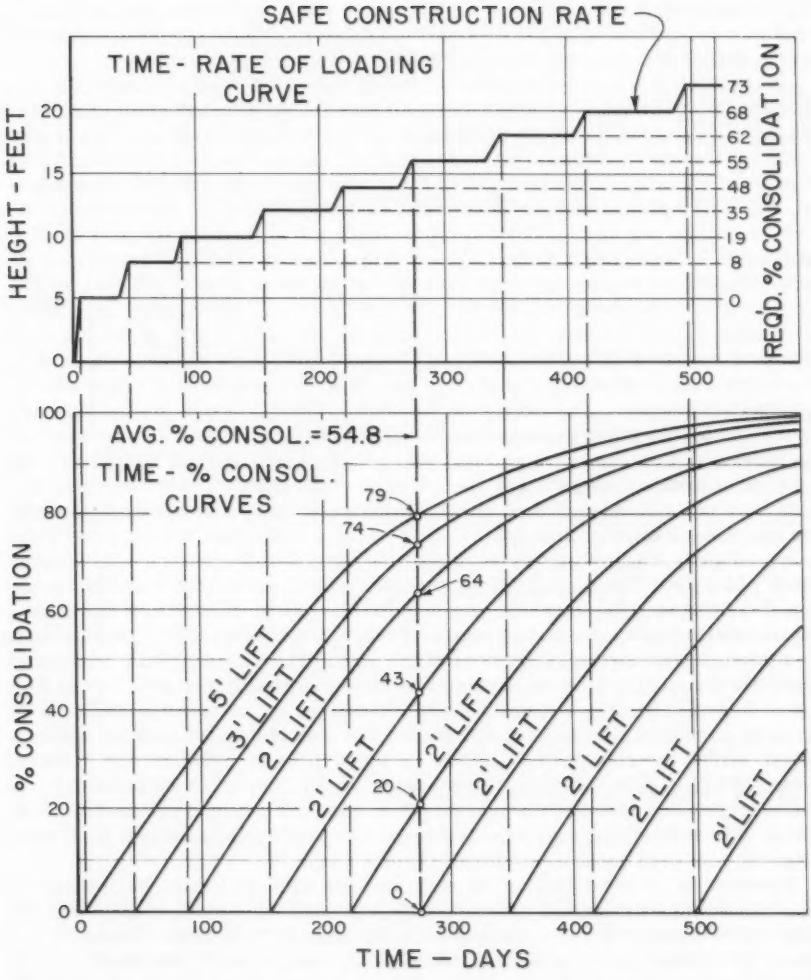

FIG. 1.—STUDY OF ALLOWABLE RATE OF CONSTRUCTION

Once the relationship between fill height and the required percent consolidation has been established, any combination of lifts can be examined without the need for further stability analyses. This procedure can simplify subsequent calculations for the "safe construction rate."

Having determined the relationship between height of fill and the percent consolidation needed for stability, and having established a representative "time-percentage consolidation" curve for the foundation, rate of loading studies can be carried out to determine the fastest, most practical construction schedule for the conditions being analyzed.

When a fill is constructed at a controlled rate, initial lifts cause a build-up of strength of the foundation that helps to support subsequent lifts. If a slowly placed fill is considered to produce the combined effect of a series of strip loadings that have been in place shorter and shorter lengths of time, the influence of each lift on the strength of the foundation can be calculated separately, and the combined effects determined by simple addition. For example, if a 5-ft increment has been in place a sufficient length of time for its portion of the foundation consolidation to be 50% completed, and another 5-ft lift is then rapidly placed, the average degree of consolidation under 10 ft of fill will be 25%. To carry out the computations for the safe rate of loading, time-percentage consolidation curves may be plotted directly beneath a plot of height of fill versus time curve as shown in Fig. 1. The required percentage consolidation, as determined by stability studies, is shown at the right on the upper plot for various fill heights. The height of fill versus time curve is plotted progressively and the weighted percent consolidation calculated from the time-percentage consolidation curves that are plotted in the lower part of the sheet. The time of placement of each lift is determined successively by "cut-and-try," with each new lift being placed at some arbitrary time, and the weighted percentage consolidation computed. If this value is less than the required value a new, slightly longer time is tried, and the weighted percentage consolidation again computed. This process is repeated until reasonable accuracy is obtained. After a little practice the time of placement of each new lift usually can be determined in one or two trials. Thus, in the example, 55% consolidation is needed at the end of placement of the sixth lift. The weighted percentage consolidation at this time is equal to 5 ft at 79% consolidation, plus 3 ft at 74%, 2 ft at 64%, 2 ft at 43%, 2 ft at 20%, and 2 ft at 0%, or an average of 54.8% for the total combined fill thickness of 16 ft. The calculated percentage consolidation under the combined influence of the 6 lifts is close to the required amount (55%). Thus, the time of placement of the last lift is satisfactory. In this manner, the safe time of placement of each lift can be determined. Following this procedure, one can study various loading patterns and determine the most practical for the conditions at each project.

Predictions of the behavior of clay masses with time must be considered approximate and one should not blindly assume that field performance will exactly follow predictions. As concluded by the author, field observations should govern the actual rate of placement of fill. Excessive pore pressures, unexplained increases in settlement, and lateral movement or heave near the toe are all warning signals that the rate is too fast.

F. P. SILVA,[3] M. ASCE.—The author is to be congratulated for his original contribution to the study of the stability of embankments resting on soft foundation soil.

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[3] Chf., Soils Sect., Instituto de Pesquisas Tecnologicas, Sao Paulo, Brazil.

In the discussion of his assumptions, the author refers to the scattering of results of permeability and coefficient of consolidation determinations. In his analysis, he adopts an average value for the coefficient of consolidation. The writer has consistently observed from consolidation tests on soft organic soils that the coefficient of consolidation decreases from the start of the test, reaches a minimum value, and then starts increasing. In the writer's opinion, the analysis might be improved further by introducing a different value of this coefficient for each increment of load.

In Fig. 1, typical pressure-void ratio (coefficient of consolidation) curves for a soft organic quaternary clay deposit near Santos, Brazil are presented. (Liquid limit for this sample was 120%; plastic limit 39%). Since these are soils of very low permeability, no increase in strength could be counted on in

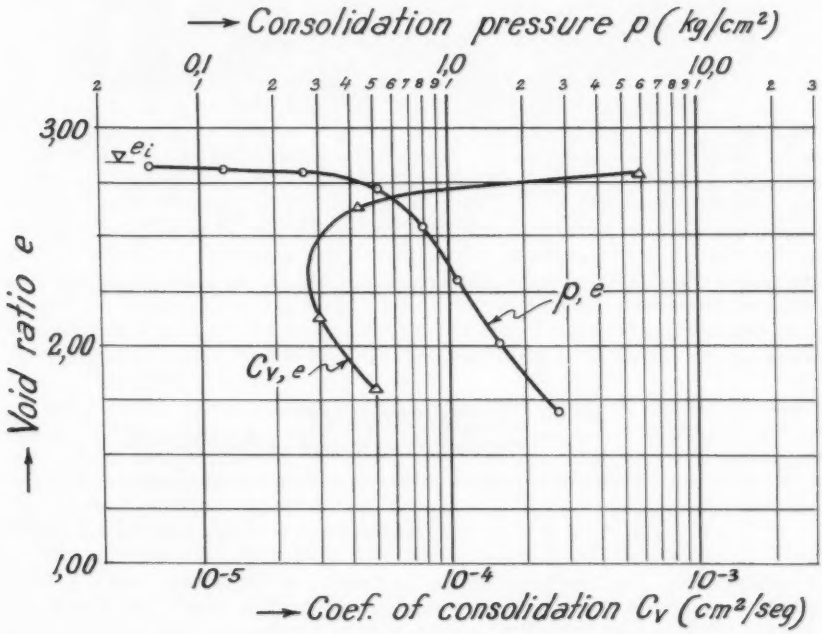

FIG. 1.—CONSOLIDATION TEST RESULTS. SOIL DEPOSIT NEAR SANTOS (BRAZIL)

any reasonable period of time, unless some means of accelerating drainage is provided, such as by the use of vertical sand drains. In this connection, coefficients of permeability on a radial direction have been determined by the writer from consolidation tests in which only radial drainage is permitted. For the purpose of this test a regular consolidation apparatus is used. The cylindrical test specimen is provided with a concentrical cylindrical drain made with a mixture of fine sand and mica powder of approximately the same compressibility as the soil. In Fig. 2, a cut of a specimen after the realization of one of such tests is presented. It is the writer's opinion that this test proves to be of great value for the determination of radial permeability coefficients in connection with vertical sand drain design.

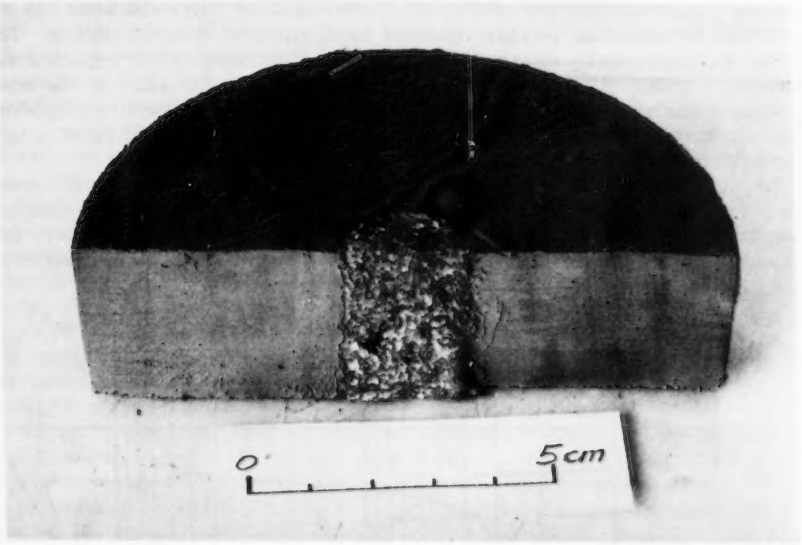

FIG. 2.—CUT OF A RADIAL DRAINAGE CONSOLIDATION TEST SPECIMEN

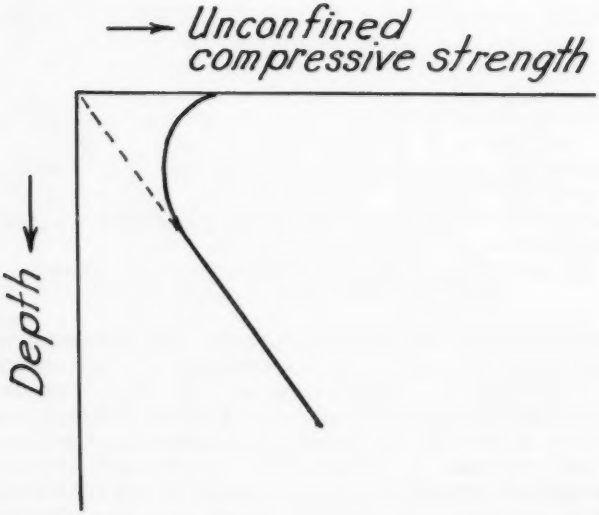

FIG. 3.—VARIATION OF UNCONFINED COMPRESSIVE STRENGTH WITH DEPTH

The writer would appreciabe knowing the author's viewpoint about trying to extend the method to the case of an initial strength condition of the type shown in Fig. 3, which is the usual case in natural soft clay deposits. Such a type of variation could, of course, be easily taken into account for the case of the first lift. Its consideration for the cases of the following lifts, however, might become too elaborate for practical use.

In connection with control piezometer installation, the author recommends double-tube type piezometers. The writer believes, however, that when the piezometer tube can be maintained in a vertical position and open to atmospheric pressure, as is the case in embankment construction, there seem to be no problems involving gas bubbles and a single-tube-type of piezometer would be preferable because of its simplicity.

O. H. GILBERT, JR.,[4] J.M. ASCE.—Mr. Lobdell has presented a very interesting and informative paper on a rational design approach to the problem of controlled loading rates of embankment filling on soft foundation soils. However, the writer feels that the author has made several oversimplifications that might lead to dangerous conclusions by the casual reader.

The author states that the effective angle of internal friction, ϕ_e, should be used when computing the shear strength of partially consolidated soils. This procedure will be conservative only when the excess pore pressures, resulting from the incremental shear stresses induced by the embankment loading, are considered. The ratio of incremental pore pressures to increase in principal stress difference (deviator stress, $\sigma_1 - \sigma_3$) is defined as the "A factor," and approaches the value of 1.0 for soft, normally consolidated clays.[5] If the pore pressures induced by shear stresses are not considered, the use of the effective friction angle, ϕ_e, is equivalent to using the drained shear strength for a given value of σ_3 (or σ_{3c}) to analyze the possibility of a sudden, or undrained, failure condition.

The strength value obtained from the effective stress envelope represents the potential strength available after complete dissipation of pore pressures induced by the increment of embankment loading. However, the strength of interest for stability is the strength immediately available at the "instant" of incremental loading. Ignoring the increase in pore pressures will lead to a safety factor very much on the unsafe side.

The necessity of estimating field pore pressures can be avoided by determining a shear strength-consolidation pressure relationship from consolidated-undrained (Qc) Triaxial Tests without pore pressure measurements. The increase of consolidation (effective) pressure with time may be determined by methods similar to those presented by the author, and the increase in undrained shear strength readily determined from the foregoing relationship. This design technique has been successfully used in the design of heavy highway fills over soft varved clay deposits in the Salt Lake City section of the Utah Interstate Highway System. The method is based upon the admittedly approximate assumption that the pore pressure behavior of the soil is similar under laboratory and field conditions. However, the writer feels that there are fewer uncertainties involved in this technique than there are in an attempt to estimate the shear-induced pore pressure in a complicated two-dimensional stress field.

The one main advantage to the effective stress approach to embankment stability is the direct correlation between available shear strength at any time

[4] Soils Engr., Porter, Urquhart, McCreary and O'Brien, San Francisco, Calif.

[5] "The Measurement of Soil Properties in the Triaxial Test," by A. W. Bishop and D. J. Henkel, Arnold, London, 1957.

and the observed field pore pressures at that time. However, as the author points out, the pore pressures observed at the embankment centerline can only be considered as a rough guide to embankment stability. It would seem, therefore, that the large number of piezometers needed in any given cross section to determine accurately the pore pressure variations along potential failure surfaces make this advantage of the effective stress concept more theoretical than practical. The writer fully concurs with the author in the desirability of locating some piezometers beneath the embankment slope, especially in areas of critical stability.

STRUCTURE AND STRENGTH CHARACTERISTICS OF COMPACTED CLAYS[a]

Discussion by I. da Silveira, O. H. Gilbert, Jr., John L. McRae

I. DA SILVEIRA,[1] M. ASCE.—Mr. Seed and Mr. Chan used a very interesting and realistic form for studying the structure effects as causes of several properties of compacted clays. The writer would like to point out another suitable method for the same purpose, that includes the factors of soil composition, in his observations and comments on the paper.

THE PATTERN BASED ON SOIL COMPOSITION INDICES

The triangular diagram was a classical form of representation of soil composition, but poor for the study of soil properties. A new system based on the voids ratio (e), the moisture content (w), and the degree of saturation (S), fill the requirement ease in construction procedures.

The compaction diagram can be replaced by a simple change of coordinates when adopting this system. Dry density (γ_d) is substituted for by the voids ratio to which it is connected by the relationship

$$e = \frac{\gamma_s}{\gamma_d} - 1 \quad \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

Since the density of solids (γ_s) is fixed and a set of saturation curves is fit so as to complete the pattern, the representation of soil having any composition is given by a point set between the curve of 100% saturation, and the curve of 0% saturation. The first corresponds to the usual zero air voids curve and the second to the e axis itself.

Fig. 1 is the basic pattern, named as (e,S,w) diagram, obtained by using a natural scale for e. Moisture content (w) values are plotted on the y axis by the adoption of a suitable scale in regards to the density ratio $y^2 = w.\gamma_s/\gamma_w$. The saturation curve is represented by the dotted line.

This procedure makes easy the construction of the (e,S,w) pattern, due to relationship among these values of

$$e\,S = w\,(\gamma_s/\gamma_w) \quad \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

A geometrical construction is based on the rectangular[2] triangle whose hypotenuse (e + S) is the x axis and the height (h) represents the w value in the suitable scale.

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[a] October, 1959, by H. B. Seed and C. K. Chan.

[1] Cons. Engr., Comissão de Solos e Fundacões, Rio de Janeiro, Brazil.

[2] This correlation is obtained from:

$$w = \frac{W_w}{W_s} = \frac{V_w}{V_s}\,\frac{\gamma_w}{\gamma_s} = \frac{S.\,V_v.}{V_s}\,\frac{\gamma_w}{\gamma_s} = \frac{S.e.V_s}{V_s}\,\frac{\gamma_w}{\gamma_s}$$

$$\text{or } e.S = w.\ \gamma_s/\gamma_w$$

The curves of equal degree of saturation are obtained by the simple geometrical procedure[3] of making S = a, or

$$y^2 = e\,a \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

Complementary curves representing the wet density (γ_m) can be plotted on the same pattern, because

$$\gamma_m = \gamma_d\,(1 + w) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

Enlarged parts of these diagrams are useful for the study of such soil properties as those connected to volume change or to any variation of dry density.

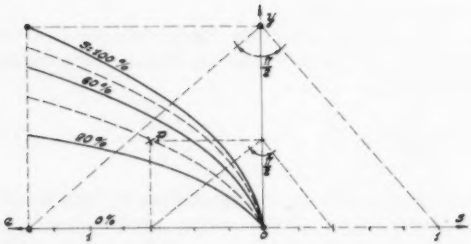

FIG. 1.—REPRESENTATION OF SOIL COMPOSITION

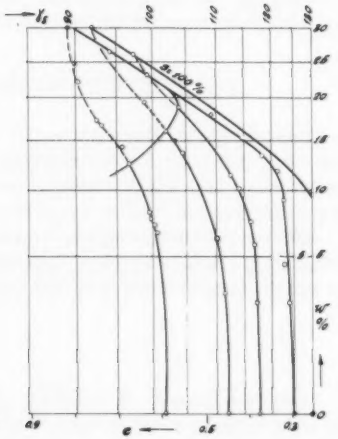

FIG. 2.—ENLARGED SOIL COMPOSITION PATTERN

The writer supposes that the representation of soaking with volume change, as referred to by the author is solved in this manner.

INFLUENCE OF STRUCTURE ON SWELLING AND ON SHRINKING

Swelling and shrinking effects were represented earlier[4] on simple compaction diagrams, but they can be done successful on the (e,S,w) pattern. For compacted soils these curves start from a point and proceed in opposite directions but form a continuous curve as shown in Fig. 2 in which the swelling part is represented by the dotted lines. Compaction curves produce a sequence of points representing different initial soil compositions.

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[3] "Diagrama de proporcao de fases para solos," by Rev. do Clube de Engenharia 193-Set, 1952, Rio de Janeiro.

[4] Road Research, Laboratory, D.S.I.R. Soil Mechanics for Road Engrs., Chapter 9, London, 1957.

Two crossed volume change curves over the same point, for a particular soil, is the picture of the effect, on different structures, of the swelling or the shrinking processes. It could be done as represented in the author's Fig. 3b, for the shrinkage process.

The lower part of the (e,S,w) pattern is able to be used for the representation of any magnitude in connection to the voids ratio. The representation of volume change of a soil, having the voids ratio (e_o) in natural scale for (-y) is a straight line given by

$$-y = \frac{\Delta V}{V} = \frac{e_o - e_1}{1 + e_o} \quad \ldots\ldots\ldots\ldots\ldots\ldots (5)$$

Axial shrinkage could be represented in this manner, but its transformation to volume change of assumed isotropic soil would be more practical, since axial shrinkage value is one third of volume change. An example of application of the lower part of the (e,S,w) pattern is the transformation of the author's Fig. 5 to Fig. 3 of this discussion. The dotted curve of Fig. 3 is the tentative curve resulting from soaked specimens.

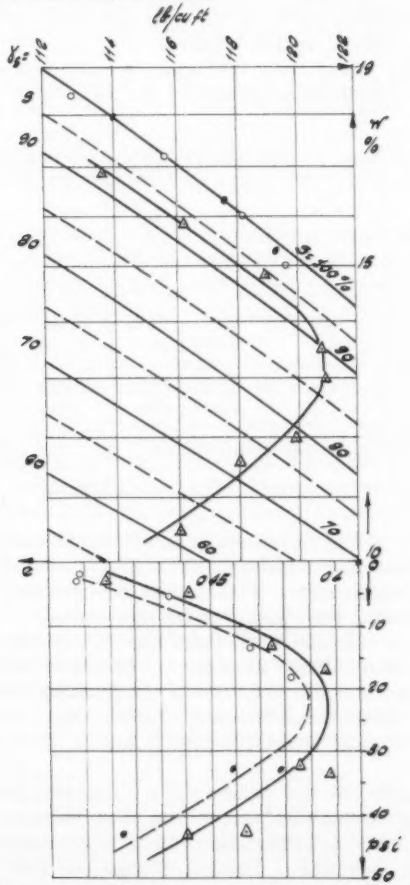

FIG. 3.—COMPACTION CURVES AND DATA

Because the voids ratio was chosen as the correlation element, the curves of swelling pressure become similar in shape to the compaction curve. The full line represents the swelling pressure for the points of the compaction curve and the dotted line is the same for soaked specimens. Soaking effect on soil structure is well represented by these separate curves. (author's Fig. 5)

The influence of molding water content on swelling pressure is quite evident and the optimum moisture point corresponds to a single point of the curve, for which the increment of that magnitude referred to the voids ratio changes the sign. Similar procedure has been used for representation of suction pressure, consolidation, and so forth.

INFLUENCE OF STRUCTURE ON STRENGTH

Soil composition indices seem to be clear when volume change effects or structure effects must be separated, even when the (e,S,w) pattern is not used.

Soil composition is particularly important when strength characteristics are examined, because it plays a recognized role on the porewater pressure as the effects of soil structure on the effective pressure are felt. Strength must be governed by both as acting together.

A table of results, taken from the author's examples was obtained for the interpretation of the questions suggested by the reading of the paper (Table 1)

TABLE 1.—ELEMENTS OF SOIL COMPOSITION

Author's Fig.	Sample	w %	e_o	S %	e_u[a]	$\frac{e_o - e_u}{1 + e_o}$	$\frac{h}{h}$
7	(1)Floc.	20.3	0.605	91.0	0.549	0.0394	0.0116
7	(2)Disp.	20.3	0.605	91.0	0.549	0.0294	0.0116
8	(1)Floc.	20.6	0.59	94.5	0.549	0.0258	0.0086
8	(2)Disp.	20.6	0.588	95.0	0.549	0.0258	0.0086
9	(1)Floc.	31.4	0.826	100	0.826	-	-
9	(2)Disp.	31.0	0.816	100	0.816	-	-

[a] Obtained by assuming the complete saturation of the sample with actual moisture content.

A general idea is that an undrained triaxial test is similar as an approach to the compaction under static load or to the consolidation of unsaturated soils. The only difference exists in the constant lateral pressure of the first test in comparision to the lateral restraining of the two others.

Water squeezing is common to all these cases, and porewater pressure is a consequence accepted as normal when the air voids become saturated. Therefore, the volume change was determined for the limiting voids ratio as is possible for undrained compressed samples, that is that corresponds to the degree of saturation for actual water content. It was obtained by making S = 1 to 2.

Stress-strain results, of the author's Fig. 7, shows two curves, one for a soaked sample having "floculated" structure, and the other for a "dispersed" structure. Soil composition is the same as an approach for both, but the degree of saturation is only 91%. Volume change for the 100% saturation is (0.605 - 0.549)/1.605 = 0.0349 and the corresponding axial strain is 1.16%.

The author's stress-strain curves (Fig. 7) show a very recognized change of slope at a strain value near this point or similar to the end of the instantaneous part of the consolidation process of a nonsaturated soil sample.

The author's two stress-strain curves have different shape, interpreted by structure effects of compacted "dry of optimum" and "wet of optimum." However the change of slope seems to have some correspondence with the point of instantaneous compression as obtained by theoretical methods due to Bishop and Hilf.[5]

The difference of slope would result from the greatest effective pressure suddenly mobilized for the "floculated" sample.

This can be presented as an effect of structure displayed through the soil composition indices (e) and principally (S).

A combined effect of voids ratio and degree of saturation can be observed from results of the author's Fig. 8 of the paper. It represents the pore water pressure diagram as having greatest value for sample number 2 that presents the smallest e(1-S) in comparision to sample no. 1. Effective stress and deviator stress are greater for sample 1 than for sample 2.

These are logical results for two soils soaked almost to the saturation but molded "dry of optimun" and "wet of optimum." The effects of unequal structure are still present.

The author's Fig. 9 deals with two samples having different initial structures and equal degree of saturation after soaking. There is not a considerable difference between the effective stresses for the samples, however, porewater pressure is greatest for the sample (number 2) having the small voids ratio, and the deviator stress is greatest for sample 1, since it follows the effective stress.

However, an acurate study of strength in compacted soils can not be completed, whether the compressibility of the skelecton or of the soil is neglected. This is the means of establishing correlations between the external acting forces and the reactions (pore water and effective pressure) that the strength depends on.

The writer recognizes again the merit of the method suggested by the authors of the paper and the valuable information contained in it.

O. H. GILBERT, JR.,[6] J.M. ASCE.—The authors are to be congratulated on an excellent and well-documented paper describing the structure and strength characteristics of compacted clays. The different structures and deformation characteristics produced by various methods of laboratory compaction seem especially significant, and should lead to modifications in both current design practice and field density control specifications.

The writer agrees with the authors in the use of the swelling characteristics of a compacted clay as an index to its structure. In a thesis supervised by the writer, J. H. Albiero[7] investigated the swell pressure behavior of a compacted Vicksburg "buckshot" clay. In the course of the investigation, two "wet-side" samples were compacted to the same water content and dry density, one by kneading compaction and the other by static compaction. The sample compacted by the static method exhibited a swell pressure three times as large as

[5] "Construction Pore Pressures in an Earth Dam," by C. Y. Li, Proceedings, ASCE, Vol. 85, No. SM 5, October, 1959.

[6] Soils Engr., Porter, Urquhart, McCreary and O'Brien, San Francisco, Calif.

[7] "Swelling Pressures of Compacted Clays," by J. H. Albiero, Unpublished S. M. Thesis, M.I.T., 1958.

that of the sample prepared by kneading compaction. These data are adequately accounted for by the authors' hypothesis of the effect of compaction shear strains on the resulting soil structure, and are in good agreement with the data presented in Fig. 14.

The investigation also showed that swell pressures determined at constant volume are not a reliable indication of structure for "dry-side" samples of low density and low degree of saturation. The water that the sample imbibes at constant volume can materially change the force pattern within the sample without material change in particle orientation. Nonetheless, this is a change in the structure of the clay, and in the case of a lightly compacted sample, the water imbibation of a "dry-side" sample can lead to a swell pressure smaller than that of the "wet-side" sample compacted to the same dry density.

The swelling tendency of a compacted clay can still be used as a structure index if changes of structure are avoided during the measurement. This can be done by measuring the pore water tensions (or negative pore water pressures) produced by compaction without any appreciable moisture transfer between the soil sample and the measuring device. This has been done in a thesis by the writer.[8] The resulting curve of pore water tension versus molding water content is similar to the swell pressure-molding water content curve presented by the authors in Fig. 5. The pore water tension technique has, as a practical limitation, tensions of one atmosphere (referred to atmospheric pressure). By careful de-airing and sealing techniques the writer was able to obtain modest absolute water tensions (up to 1.3 atmospheres) with a sealed electric transducer pore water pressure device, but these tensions were unstable and difficult to reproduce consistently.

The authors have concluded that soil structure has little or no effect on the strength of compacted clays in the "as-compacted" condition when the strength characteristics are expressed in terms of effective stresses. Had the authors qualified this conclusion by limiting it to samples which had been soaked to near saturation under a confining pressure, the writer would be in full agreement. Data by Huning[9] on a compacted Venezuelan clay, obtained by essentially the same testing technique as that used by the authors, leads to the same conclusions. Davison[10] tested soaked, compacted samples of a silty clay to which trace chemicals were added in an attempt to change the soil structure. Although the expected changes in structure were observed, all samples fitted the same strength envelope when plotted in terms of effective stresses.

However, the process of soaking changes the structure of the sample from the initial structure of the "as-compacted" state. Generally speaking, the increase of water content changes the soil sample from very brittle to a rather plastic material.

The writer has attempted to determine the effective-stress strength parameters of a compacted silty clay in the "as-compacted" condition by measuring the negative pore water pressures of unconfined samples during shear. The material tested was a Vicksburg Loess (L.L. = 34, P.L. = 26, minus .002 = 14%) compacted by kneading compaction. The results of the test series is shown in Fig. 1. In Fig. 1(a) the stress paths of the samples in terms of effective

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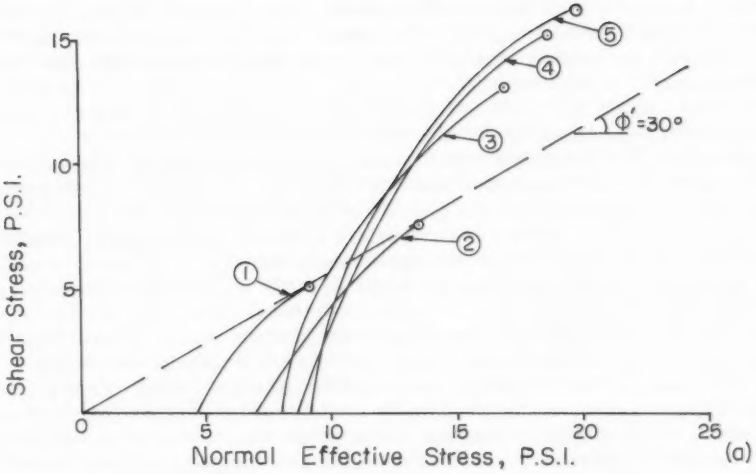
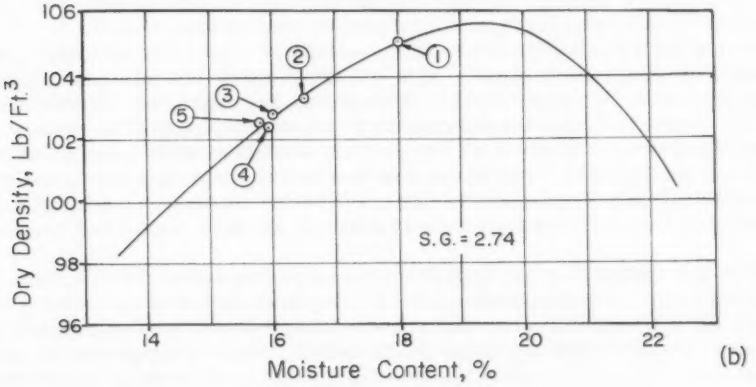
[8] "The Influence of Negative Pore Water Pressures on the Strength of Compacted Clays," by O. H. Gilbert, Unpublished S. M. Thesis, M.I.T., 1959.

[9] "The Effect of Structure on the Strength of Compacted Soil," by H. C. Huning, Unpublished S. M. Thesis, M.I.T., 1957.

[10] "The Effects of Aggregants and Dispersants on the Strength Characteristcs of Compacted Soil," by D. M. Davison, Unpublished S. M. Thesis, M.I.T., 1959.


Shear Stress, P.S.I.
15
10
5
0
5
10
15
20
25
Normal Effective Stress, P.S.I.
Φ'=30°
1
2
3
4
5
(a)


Dry Density, Lb/Ft.3
106
104
102
100
98
96
14
16
18
20
22
Moisture Content, %
S.G. = 2.74
1
2
3
4
5
(b)

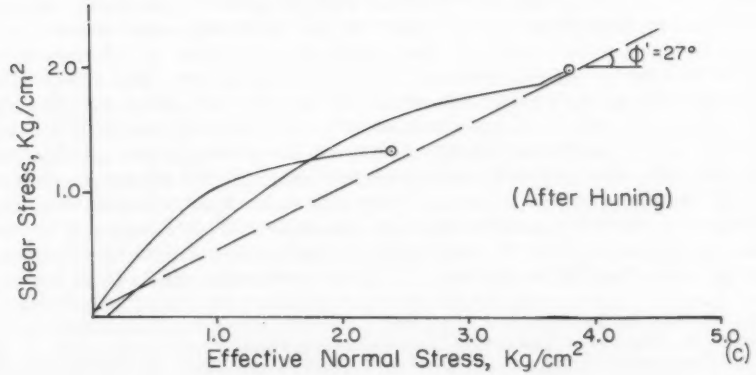

Shear Stress, Kg/cm2
2.0
1.0
1.0
2.0
3.0
4.0
5.0
Effective Normal Stress, Kg/cm2
Φ'=27°
(After Huning)
(c)

FIG. 1

stresses have been plotted as vector curves. (All samples failed in a brittle fashion at less than 2.4% strain). The dotted envelope shown on this plot was obtained from conventional triaxial tests in which the samples were soaked and back-pressured to saturation.

It can be seen that the two tests closest to optimum agree with the soaked envelope while the three other tests rose well above the envelope.

It is the writer's hypothesis that the ability of the samples to rise above the failure envelope is a result of strong attractive links between particles, present in the initial structure of the sample. As shearing continues, the initial structure is broken down. If the initial structure of the sample is plastic enough to allow continuous deformation to large strains, the effects of initial structure are eradicated by the time the ultimate strength is attained and the sample strength may be unique in terms of effective stress. This conclusion is borne out by the results of Huning. For sake of comparison vector curves from two of his back-saturated, back-pressured unconfined tests are shown in Fig. 1(c). For these tests the vector curves rise above the failure envelope at low strains. As shearing continues to higher strains, negative pore pressures of sufficient magnitude are developed to bring the vector curve back to the failure envelope.

In conclusion, there is considerable evidence to suggest that there may be a unique relation between shear strength and effective stresses under conditions leading to plastic failure: wet-side compaction, high confining pressures, water content change after compaction by ground-water permeation.

The conditions under which long term stability of most earth structures are investigated correspond closely with the latter case. In this case, shear strength studies, in terms of effective stresses, are desirable. On the other hand, for studies of the short time stability (construction period) of moderately low structures with dry-side compaction, effective stress analysis offers no more advantages than total stress analysis in the light of existing knowledge. The most important problem, in this case, is the selection of a laboratory compaction method that closely corresponds with the field compaction method.

JOHN L. McRAE.[11]--It is highly gratifying to see the increased attention being given to the very important matter of compacted soil structure. In 1944, certain work in the Corps of Engineers Soils Division of the Waterways Experiment Station, Vicksburg, Miss., indicated that some of the phenomena observed in our work with soil compaction and the California bearing ratio (CBR) test could only be explained in terms of compacted soil structure. At that time, the more immediate problem was to get soils engineers to recognize that certain phenomena could only be explained on the basis of differences in soil structure rather than to advance a theory to explain why. The group with which the writer worked advanced a theory at that time which we thought gave logical support to the contention that there was a difference between the arrangement of soil particles for different methods of compaction and for soil compacted on the dry side of optimum from the arrangement of particles compacted on the wet side of optimum. This argument was presented[18] in the Waterways Experiment Station technical memorandum.[12] In essence, it postulates an arrangement of water, air, and soil that is more uniformly dispersed on the wet side than on the dry side and in which the more the soil is worked

11 Engr., Chf., Bituminous and Chem. Sect., Flexible Pavement Branch Soils Div., US Army Engr. Waterways Experiment Sta., Vicksburg, Miss.

12 "The California Bearing Ratio Test as Applied to the Design of Flexible Pavements for Airports," No. 213-1, July 1, 1945.

by the compaction process, the more homogeneous is the dispersion. The attempt was primarily to show, in the CBR report, that the problem did exist and an explanation was needed.

Around 1948, while the writer was a research associate in soil mechanics at Northwestern University with P. C. Rutledge and J. O. Osterberg, some further experimental evidence was obtained with regard to soil structure that had not been previously published. It is believed this evidence lends further support to Mr. Lambe's theory of a "flocculated" dry-side structure and a "dispersed" wet-side structure, but it also indicates that the concept needs to be modified or expanded, particularly the theory for the wet-side structure.

The experimental evidence referenced above is presented in the photographs on Figs. 1 and 2. These are photographs of chunks of silty clay soil that have been compacted in the laboratory by various compaction procedures. In Fig. 1 (a) W = 10.9%, P_d = 99.4 lb per cu ft; in Fig. 1 (b) W = 22.7%, P_d = 102.8 lb per cu ft; in Fig. 2 (a) W = 10.1%, P_d = 116.8 lb per cu ft; in Fig. 2 (b) W = 17.1%, P_d = 109.7 lb per cu ft. The silty clay soil of Fig. 2 was compacted under 2000 psi static load, while that of Fig. 1 was compacted by kneading static 150 psi foot pressure. The kneading static compaction referred to was accomplished with the Northwestern kneading compactor[13] and is essentially the same kind of compaction that Mr. Seed refers to as "kneading compaction." The exposed faces of these chunks of soil were obtained by breaking the soil open, as cutting would not demonstrate the texture of the exposed soil. Fig. 1 (a) and (b) show the internal structure of silty clay compacted by the kneading method dry of optimum and wet of optimum, respectively. Fig. 1 (c) and (d) show the effect of soaking chunks of dry-side and wet-side material in water, respectively. It is evident from visual observation that there are marked differences between the soil chunks representing the dry side and the soil chunks representing the wet side. Note that on the dry side of optimum (Fig. 1 (c)) there is a somewhat smoother texture and an appearance of disintegration on an individual grain basis. This seems to be in harmony with Mr. Lambe's flocculated arrangement concept.

On the wet side, however, there is a definite layered or laminated texture. This laminated texture is more evident to the naked eye than these photographs of the unsoaked specimens would indicate. However, it is quite evident in the soaked chunks where the thin flakes and layers are readily seen from the disintegration in the soaking process. Mr. Lambe's dispersed wet-side concept may be essentially correct for the structure within the laminations or layers, but some further explanation appears needed to describe the over-all structure.

The wet side seems to be further characterized by the formation of thin laminations or sheets within which there is a bond retained between the particles that resist disintegration when immersed in water, but the sheets or laminations do not retain a bond to each other but readily separate when immersed in water. Also, it is important to note that the wet-side specimens are often characterized by spongy or springy properties. Fig. 2 shows the same thing for silty clay compacted by static loading. This static loading was accomplished with a single plunger only slightly smaller than the diameter of the test mold. There was some plastic flow under the 2000-psi load, however, as evidenced by soil squeezing out around the plunger.

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13 "Laboratory Kneading of Soil to Simulate Field Compaction," by J. L. McRae, and P. C. Rutledge, Proceedings of 31st Annual Meeting of the Highway Research Bd., 1952.

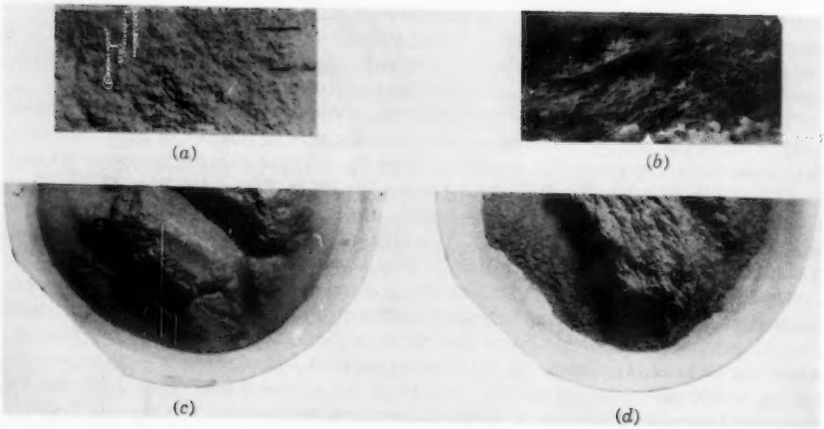


FIG. 1

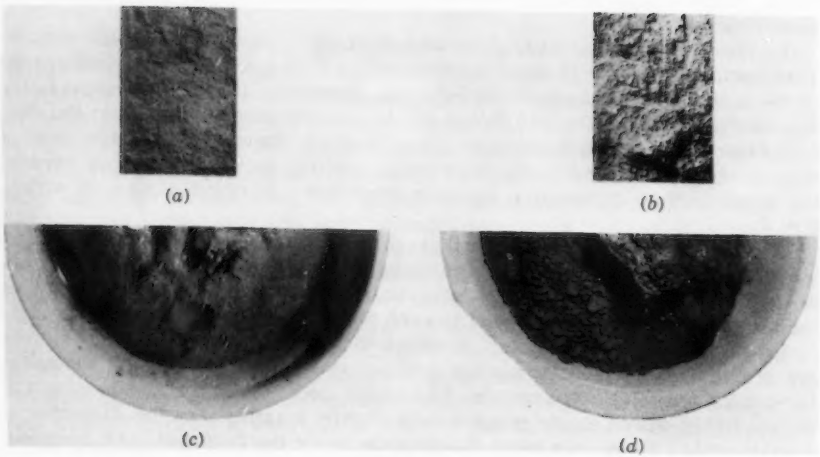


FIG. 2

The Corps of Engineers now has, as a matter of practical necessity, grouped[14] subgrade and base course soils into the following three classes with respect to behavior in the CBR test

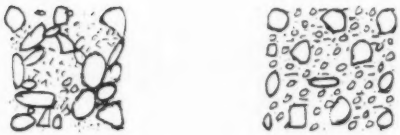
a. Cohesionless sands and gravels (GW, GP, SW, and SP).
b. Cohesive soils (GM, GC, SM, SL, ML, CL, and OL).
c. Highly swelling soils (MH, CH, and OH).

Soils in group a show no difference in CBR test properties between wet side

Dry Side Wet Side

GROUP A (GW, GP, SW, SP)

Soil structure not affected by initial water content or method of compaction. Never develops reverse CBR stress-strain curve.



Dry Side Wet Side

GROUP B (GM, GC, SM, SC)

Soil structure may be affected by initial water content and method of compaction. May develop reverse CBR stress-strain curve.

Dry Side Wet Side

GROUP B (ML, CL, OL)

Soil structure may be affected by initial water content and method of compaction. May develop reverse CBR stress-strain curve.

Dry Side Wet Side

GROUP C (MH, CH, OH)

Soil structure may be affected by initial water content and method of compaction. No cases of reverse CBR stress-strain curve has been observed.

FIG. 3.—COMPACTED SOIL STRUCTURE

[14] EM 1110-45-302, p. 33, dated August, 1958.

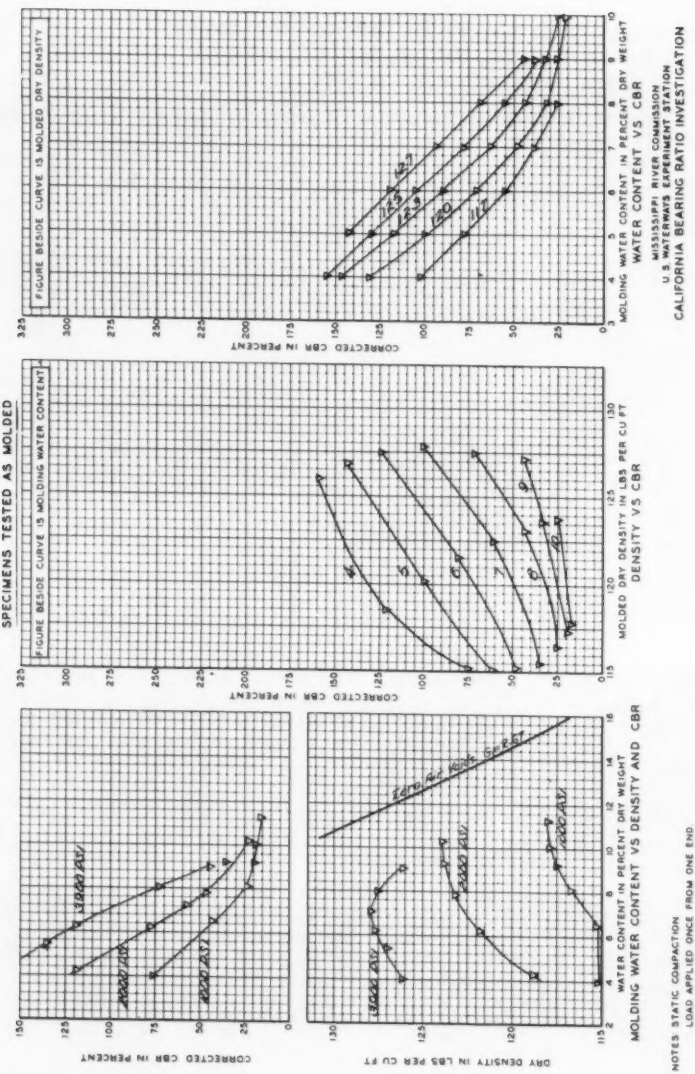

SPECIMENS TESTED AS MOLDED
FIGURE BESIDE CURVE IS MOLDING WATER CONTENT
FIGURE BESIDE CURVE IS MOLDED DRY DENSITY
CORRECTED CBR IN PERCENT
MOLDED DRY DENSITY IN LBS PER CU FT
DENSITY VS CBR
MOLDING WATER CONTENT IN PERCENT DRY WEIGHT
WATER CONTENT VS CBR
MISSISSIPPI RIVER COMMISSION
U.S. WATERWAYS EXPERIMENT STATION
CALIFORNIA BEARING RATIO INVESTIGATION
MOLDING WATER CONTENT VS DENSITY AND CBR
DRY DENSITY IN LBS PER CU FT
WATER CONTENT IN PERCENT DRY WEIGHT
NOTES STATIC COMPACTION
LOAD APPLIED ONCE FROM ONE END
SURCHARGE: 10 LB PENETRATION

FIG. 4.—VARIATION OF CBR WITH DENSITY AND WATER CONTENT

and dry side or for different methods of compaction. Soils in group b are liable to show marked differences between wet side and dry side and for different methods of compaction. Soils in group c are not extremely sensitive, by the CBR test, to molding water content in the as-molded condition but do present a problem from the standpoint of swell, and the water content and density must be selected at such a combination as to minimize expansion when the soils take up water.

The writer believes the theoretical soil structure for wet side and dry side for these three soil groups could be pictured as in Fig. 3. This picture could be modified as to degree of dispersion to account for variations in the amount of shear strain when using various methods of compaction.

With regard to comparative strengths by different methods of compaction, the author stated that no doubt exceptions could be found to his findings with the silty clay used in his tests. This is quite true. Fig. 4 is a notable exception in which dynamically compacted and soaked (clay sand) CBR specimens show higher CBR values at the lower water contents with rapid reduction in CBR as the water content increases. This is directly opposite to the trend shown by statically compacted specimens.

In conclusion, attention is called to the fact that a somewhat parallel situation with regard to structure exists in compacted bituminous mixtures. This has been brought to light be Messrs. C. L. Monismith and B. S. Vallerga.[15] It would then appear desirable to attempt to extend the structure theory eventually to apply to granular plastic materials, in general.

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15 "Relationship Between Density and Stability of Asphaltic Paving Mixtures," by C. L. Monismith and B. A. Vallerga, Proceedings of the Assn. of Asphalt Paving Technologists, Vol. 25, February, 1956.

DESIGN OF UNDERSEEPAGE CONTROL MEASURES FOR DAMS AND LEVEES[a]

Discussion by Karl H. Evans

KARL H. EVANS,[1] M. ASCE.—Messrs. Turnbull and Mansur should be commended for a thorough and complete analysis of the present status of underseepage control measures.

The writer wishes to question the statement "Seepage control measures are considered necessary where observed or estimated values of h_δ^* may be expected to equal or exceed h_c (approximately 0.75 z_t) at flood stages." Later the authors state "Seepage berms should have a width and thickness such that i through the top stratum and berm at the landside toe of the levee will not exceed 0.5 and i at the berm toe will not exceed 0.75 to 0.80." From this, it is surmised that the authors are recommending the use of a critical gradient through the blanket material and are recommending a factor of safety of approximately 1.5 at the landside toe of the levee.

Exploration data along the Missouri River northeast of Atchison, Kansas made available to the writer indicates that the critical gradient for a material normally classified as a fat clay may vary from 0.85 to 0.61; medium clay, 0.85 to 0.54; silty clay, 0.83 to 0.75; and silt, 0.95 to 0.79. These materials were encountered from a depth of 0 ft to 14 ft from the surface and within a length of about 8 miles. These results were obtained from a total of 57 soil samples. This information is put forward to illustrate that the use of an average gradient of 0.75 may, in some instances, be considerably in error depending on the composition of the top stratum.

A question also arises as to the use of a critical gradient as an indication of a need for pressure relief. The derivation of the expression for critical gradient ($i_c = \frac{G-1}{1+e}$, in which i_c is the critical gradient, G is the specific gravity of soil particles, and e is the void ratio of the soil) is available in most Soil Mechanics textbooks. This expression for critical gradient can be obtained by setting the intergrannular pressure (or effective pressure) between soil grains equal to zero. Thus, it is reasoned that, since the intergrannular pressure is zero, there will be no frictional resistance to the movement of the soil grains and they can be carried upward by the flow of water. It appears that in a clay or silty soil the cohesive strength would give added resistance to movement and that the use of a critical gradient as a criterion would be a conservative assumption. However, this may not be true.

The writer is of the opinion that the safety, with respect to uplift, should be investigated. The uplift safety factor may be defined as the ratio of the saturated weight of the blanket to the hydrostatic pressure at the base of the blanket. Fig. 1D illustrates the relationship between the critical gradient safety

[a] October, 1959, by W. J. Turbull and C. I. Mansur.

[1] Assoc. Prof. of Civ. Engrg., Univ. of Missouri, Columbia, Mo.

factor and the uplift safety factor, assuming the soil to be 100% saturated. To illustrate the use of this chart, let us assume a critical gradient of 0.75 and an actual gradient of 0.5. The factor of safety, with respect to gradient, is then $\frac{0.75}{0.5}$, or 1.5. Entering the chart with a gradient though the blanket of 0.5 and reading vertically upward to a critical gradient of 0.75, it is seen that the ratio of gradient safety factor to uplift safety factor is 1.25. Thus, the uplift safety factor is 1.20. It can also be shown by Fig. 1D that if the gradient safety factor is greater than 1.0, the uplift safety factor is also greater than 1.0, and the uplift safety is always less than the gradient safety factor. It should be cautioned

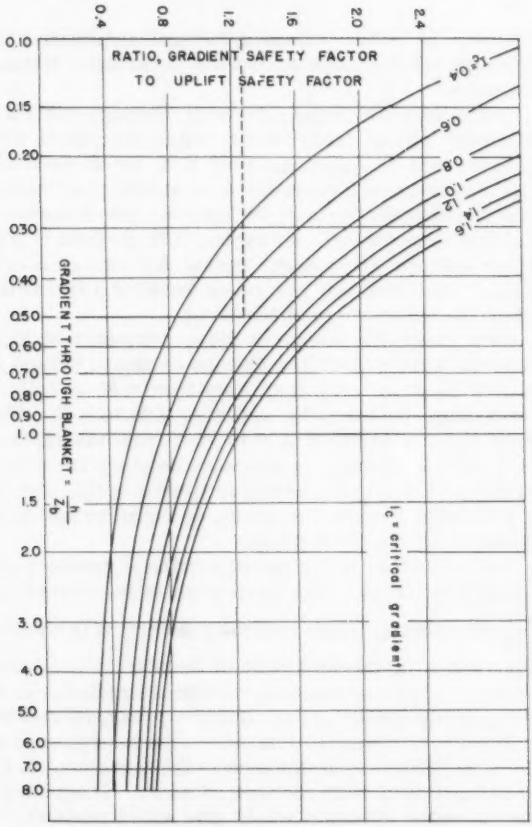

FIG. 1D.—COMPARISON OF SAFETY FACTORS

that this is true only if the soil blanket is completely saturated. If the degree of saturation is less than 100%, it is possible to have a gradient safety factor greater than 1.0 and an uplift safety factor less than 1.0.

Preliminary laboratory tests indicate that it is extremely difficult to obtain 100% saturation of a clay or silt blanket. The experiments also indicate that there is frequently a tendency for the blanket to lift off the foundation sand. When this occurs the bottom of the blanket will slough off and the blanket will

get progressively thinner until it is no longer capable of resisting the uplift pressure. In experiments in which the blankets were prevented from rising, the pressure necessary to cause failure was greater than that computed from the critical gradient criterion, indicating that the soil blanket would have been lifted up before the critical gradient pressure could have been reached.

The writer is of the opinion that the factor of safety with respect to uplift is a more logical approach than the use of the critical gradient which was derived for a sandy type of soil (essentially, a non-cohesive soil).

COMPACTION OF SANDS AND BEARING CAPACITY OF PILES[a]

Discussion by S. C. Schiff, Charles Szechy, B. A. Kantey, A. A. Eremin, Yoshichika Nishida, W. I. Low

S. C. SCHIFF,[1] A.M. ASCE.—In a series of experiments, still in progress (1960), the writer has found a behavior pattern for a group of test piles in sand, which sheds some additional light on the subject so excellently treated in the paper.

The following data represents the results of several hundred tests:

1. The rupture load of any one pile in a group is considerably above that carried by a single pile.
2. The center to center distance between pile axes plays a cardinal role in the rupture-load attained; the limit of effectiveness seems to be quite beyond that prescribed by Mr. Meyerhof. Tests went as far as 12 diam apart and had not, by far, reached the limit.
3. For equal penetration the piles in the groups carry 1 to 7 times higher per pile load than a single pile, the range depending on the variable tested—each value being corroborated by several tests. This is in brusque disagreement with the experimental values referred to by the author. This difference can be explained by the heterogeneity of the soil used in the experiments which included among others a substantial clay layer. It is, also, to be noted that the center to center distances used in those experiments are 2 and 3 times the diameters which have been established by the writer as most inefficient distances for pile groups.
4. It should also be noted that applying the full rupture load of a single test pile to each pile in a group, brought about group settlements varying between 1/2 to 1/50 of that of the single pile.

Piles tested were 25 mm x 25mm, 1-meter-long, solid steel piles, dynamically driven to a predetermined depth and then statically loaded up to 12 tons per group. The number of piles in a group varied from 2 to 5. Tests in preparation are to be in the 100 ton range on penetrometric piles, thus separating point bearing from friction load values. Tests were performed in a dry Seine River sand.

CHARLES SZECHY.[2]—Mr. Meyerhof has concluded that the bearing capacity of piles driven into cohesionless soils when calculated from static formulas—is very often erroneous because the influence of compaction and prestressing of the soil by pile driving is, generally, ignored. In the writer's experiments,

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[a] December, 1959, by G. G. Meyerhof.

[1] Ingéieur Stagiaire, Service Sol et Foundation, C.E.B.T.P., 12, rue Brancion, Paris 15, France.

[2] Prof., University of Civ. Engrg. and Architecture, Budapest, Budapest, I. Roham u.3.

executed at the University of Civil Engineering and Architecture, in Budapest, it has also been found that both factors, extrusion of the soil mass equal to the volume of the pile, and vibration, produced by driving itself, are bringing about a considerable compaction effect in the neighborhood of the pile leading in turn to a considerable increase of the soil resistance properties and, consequently, to an increase of the bearing-capacity of the pile.

Based on these experiments, the writer has set up a new pile-bearing formula, in which this effect is, approximately, taken into account, and where the other defect of static formulas—namely the direct addition of the two bearing components (mantle friction and tip resistance) irrespective of their correlation and influence to each other,—is also eliminated.

We may assume that in the limit condition of equilibrium, Rankine's ratio is valid between the vertical stresses set up below the toe, q_u, and the lateral stresses, p, acting upon the shaft-mantle of the pile. Thus, the general formula:

$$R = q\,A + f\,S \quad \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

may be written for a cylindrical pile as

$$R = q_u\,r^2\,\pi + f\,2\,r\,\pi\,D\,p_a \quad \ldots\ldots\ldots\ldots\ldots (2)$$

in which q_u is the ultimate unit base resistance, p the unit normal stress on mantle surface, r the radius and D the length of pile, and f denotes the average unit shaft friction factor.

At pile-tip level, it was assumed that $p = K\,q_u$ and with a parabolic law of normal stress distribution $p_a = \frac{2}{3}\,n = \frac{2}{3}\,K\,q_u$ (Fig. 1).

After substitution we get

$$R = q_u\,r^2\,\pi + f\,2r\,\pi\,D\,\frac{2}{3}\,K\,q_u = r^2\,\pi\,q_u\left(1 + \frac{4}{3}\,\frac{D}{r}\,f\,K\right) \ldots\ldots (3)$$

In this equation the value of f is increased by the effected compaction. This compaction is produced partly by vibration and partly by the volume of the driven pile shaft which extruded an equivalent soil-mass in its surroundings. The degree of compaction is biggest at the pile shaft surface and is decreasing with the distance from it. It may be assumed that the decrease in void content Δn, may be a measure of this compaction and this is also decreasing with the distance. The percentage decrease, Δn_a, in void content will be obtained, when the volume of driven pile shaft is related to the surrounding larger diameter cylinder influenced by compression. The bigger the zone of influence the less this average decrease in void content will be.

This may be written

$$\Delta n_a = \frac{r^2\,\pi\,D}{\alpha\,r^2\,\pi\,D\,(1 - n_o)} = \frac{1}{\alpha^2\,(1 - n_o)} \quad \ldots\ldots\ldots\ldots (4)$$

in which $\alpha\,r$ is the radius of influence-cylinder and r is the radius of pile shaft, n_o is the original percentual void content and Δn_a refers to the average change

of void content of soil. This indicates a concave parabolic ratio and is represents, also, the fall of stress intensity. The degree of compaction as a function of normal stresses is decreasing, however, according to Prof. Meyerhof's experiments, very moderately from the peak pressure on the shaft and following a logarithmic rule. Therefore, it seems to be justified to assume that the actual change of void content will decrease linearly with the distance from the pile shaft. This means that its peak value (Δn_{max}) on the pile shaft may be taken as the double of the average value Δn_a. Several tests have shown that

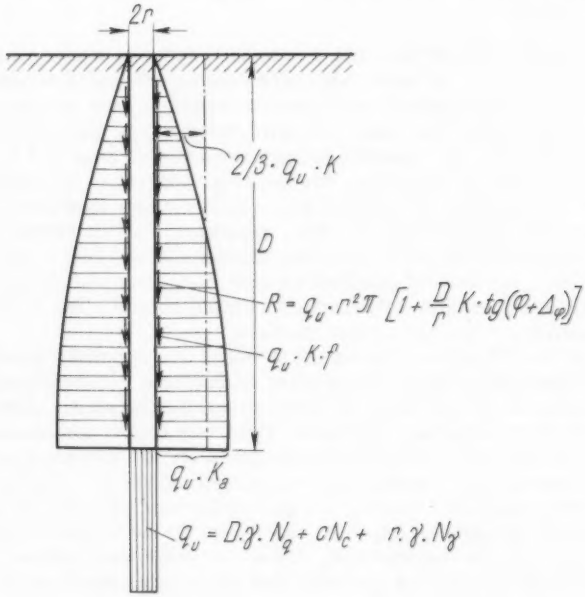

FIG. 1

within practical limits the increase of the inner friction value in a cohesionless soil is also a linear function of the decrease of void content; in sand $\Delta\rho_s^o = \frac{\Delta n_u}{2}$ % and in sandy-gravel $\Delta\rho_{gr}^o = \frac{\Delta n}{4}$.

In the writer's calculations, it is possible to obtain the value Δn_a very easily from Eq. 4, if we assume a practical value for α lying between 4 and 6, depending upon the original density of the cohesionless soil. Considering that mantle-friction will be dependent upon $\Delta n = \Delta n_{max}$ and $\Delta n_{max} = 2\,\Delta n_a$, we get that the increase of the angle of inner friction at the shaft surface will be $\Delta\rho_s = \frac{\Delta n_{max}}{2} = \frac{2\,\Delta n_a}{2} = \Delta n_a$ % and $\Delta\rho_{gr} = \frac{\Delta n_a}{2}$ %

Returning now to our equation 3 the value of f will be increased by $\Delta\rho_s$ or $\frac{\Delta\rho_{gr}}{2}$, respectively, and we may write

$$R_s = r^2 \pi q_u \left[1 + \frac{4}{3}\frac{D}{v} K \operatorname{tg}\left(\rho_s + \Delta\rho_s\right)\right] \ldots\ldots\ldots\ldots (5)$$

for sands and

$$R_{gr} = r^2 \pi q_u \left[1 + \frac{4}{3} \frac{D}{r} K \operatorname{tg} \rho_{gr} + \frac{\Delta \rho_{gr}}{2} \right] \ldots\ldots\ldots\ldots (6)$$

for sandy gravels, which is representing a very simple improved static pile-bearing formula, where compaction effect and the relative influence of both factors—tip-resistance and mantle friction—on each other are approximately taken into account.

B. A. KANTEY,[3] M. ASCE.—The author has produced a paper that confirms the impression formed by most engineers dealing with piled foundations; that driving piles into cohesionless soils must compact the soil and hence increase the bearing capacity of the soil. He puts forward the proposition that, for driven piles, the bearing capacity is approximately doubled and for displacement piles of the Franki pile type, the bearing capacity is increased approximately sixfold. However, the results given by the author indicate that the zone in which the bearing capacity is, thus, increased can be limited to a lateral distance of one diameter and a horizontal distance of one and a half diameters below the piles. The fall off in effect is such that, even with the displacement pile, the bearing capacity is only doubled within a zone limited to a distance of twice the diameter of the bulb, from the bulb.

Of primary importance is the extent to which one can make practical use of the author's findings in the design of piled foundations. Simplifying the problem for the moment to the case of 18-in-diam. displacement piles of 60 tons capacity, driven at a spacing which will effectively double the measured bearing capacity of the soil within the limits given above, there appear to be two alternative approaches. Firstly, one could determine the depth at which the soil would safely support a loading per pile of 60 tons, assume that driving the piles will double the bearing capacity and design on the basis of 120 tons per pile. This approach is impractical, since the piles themselves will not be capable of supporting 120 tons per pile. The alternative would be to determine that depth at which the soil will safely support 30 tons per pile, assume that driving of the piles will double the bearing capacity as before and design on the basis of 60 tons per pile. In this case, however, it would appear that this solution will be ruled out on the grounds of adverse settlement.

While it is true that the preceding is an oversimplification of the problem, it does point the difficulties involved in attempting to adapt the author's theories to practical design purposes. In many piling problems, one is faced with a great depth of loose cohesionless soil in which it would be most advantageous to be able to use a shallower founding depth at a higher bearing capacity than that indicated by penetration tests. In effect, the author rules such a solution out on the grounds that, although closely spaced piles could so compact the soil as to make the group capacity as much as the sum of the base resistance of an equivalent block footing and the skin friction on the block, less the weight of soil enclosed, this increased bearing capacity cannot be utilized due to settlement considerations. The writer is of the opinion however, that it might be more advantageous to pursue the effect on skin friction of this increase in compaction and, hence, the distribution of loading through the pile group to the soil below. If it can be shown that the effect of driving the piles into cohesionless

[3] Cons. Engr., Cape Town, South Africa.

soil is not only to increase the bearing capacity in the neighborhood of the base of the pile, but also so as to increase the skin friction that a large portion of the total loading is removed from the base and carried in the compacted upper zones, then the settlement problem might be brought down to acceptable limits. The writer would very much appreciate the author's views on the design possibilities inherent in this approach.

In South Africa, the most popular method of testing piled foundations is by the use of the static cone penetrometer. Judging from the values given by the

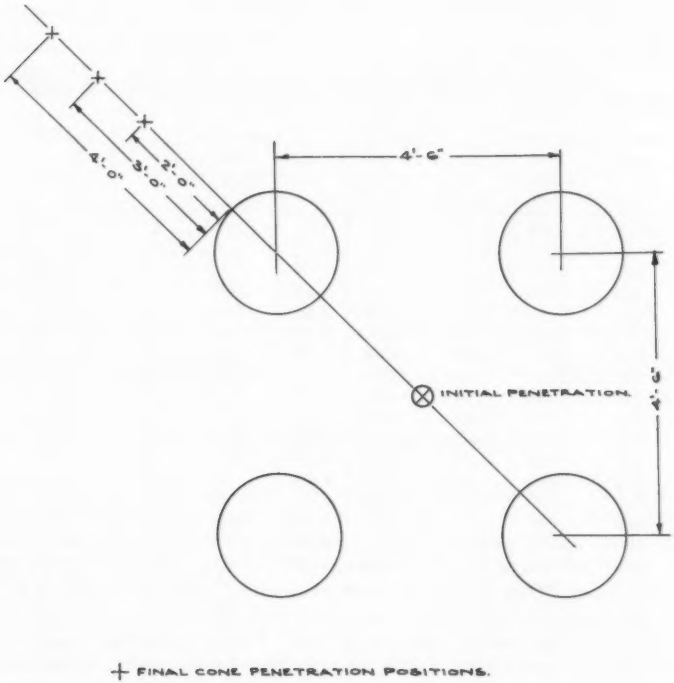

FIG. 1

author for the cone penetrometer resistances of two of his examples, local soils appear to be in a much looser state than those investigated. Fig. 1 shows a group of Franki displacement piles of 22 in. diam each, driven to refusal and Table 1 gives the results of the cone penetration test carried out at the positions shown. Examination of the results of the initial penetration tests shows that at depth, the values obtained are generally less than the values of 50 to 100 ton per sq ft (700 to 1400 psi) quoted by the author and, consequently, it is found that the ratio of final to initial resistance is greater than that found by the author. While this tends to confirm the author's statement that the increase in relative density is much greater for loose sands than for denser sands, it is also apparent that this must have a marked affect on the ratio of final to initial penetration resistances. Fig. 2 shows the results of the initial

penetration resistances and the ratio of final to initial penetration resistances at a distance of 3 ft from a pile, and in the middle of the group of four piles. The soil profile consists of approximately 20 ft of hydraulically placed sea sand, overlying the original sea bed sands. Marked differences in the pattern of results are found from those reported by the Author. First and foremost, the initial penetration results would indicate a design depth of founding of 42 ft by local practice. The piles however, were driven to required set at a depth of 38 ft. This would tend to give some confirmation to the theories of the author,

TABLE 1.—RESULTS OF CONE PENETRATION TESTS

Depth ft.	Initial	After Driving Piles.							
		In Middle of Group		2 feet from Pile		3 feet from Pile		4 feet from Pile	
	Results in pounds per square inch	Results in pounds per square inch	Ratio	Results in pounds per square inch	Ratio	Results in pounds per square inch	Ratio	Results in pounds per square inch	Ratio
9	1700	3800	2.24	2800	1.65	2700	1.59	3500	2.00
10.5	550	2800	5.10	1800	3.26	1400	2.55	2000	3.64
12	500	2400	4.80	1500	3.00	1400	2.80	1500	3.00
13.5	325	2400	7.38	1500	4.61	2000	6.15	1200	3.70
15	1100	3000	2.73	2500	2.28	2200	2.00	2400	2.18
16.5	800	2500	3.13	2200	2.75	2000	2.50	1800	2.25
18	650	1600	2.46	1300	2.00	1200	1.85	1600	2.46
19.5	900	2600	2.90	2200	2.40	1700	1.90	1600	1.84
21	1000	3400	3.40	3000	3.00	2000	2.00	1200	1.20
22.5	600	2200	3.67	2600	4.33	2600	4.33	1900	3.17
24	600	2800	4.67	2900	4.83	2000	3.33	2000	3.33
25.5	400	1500	3.75	1900	4.75	1200	3.00	1300	3.25
27	350	2000	5.71	1000	2.85	1400	4.00	600	1.71
28.5	600	2200	3.67	600	1.00	1000	1.67	1000	1.67
30	400	1600	4.00	1400	3.50	1400	3.50	1300	3.25
31.5	400	3000	7.50	2400	6.00	2300	5.75	1000	2.50
33	125	6000	48.00	8000	64.00	1600	12.80	800	6.40
34.5	225	800	3.55			600	2.66	2000	8.90
36	125	1000	8.00			1250	1.00	1000	8.00
37.5	350	1200	3.43			1600	4.57	500	1.43
39	525	900	1.71			1000	1.90	1200	2.29
40.5	500	800	1.60			500	1.00	1300	2.60
42	1500	1400	1.00			1500	1.00	1600	1.00
43.5	3200								
45	4000								

in that it can be assumed that the lower 4 ft between 36-ft and 42-ft depths were sufficiently compacted to give the required bearing capacity. However, the penetration resistances, after driving, indicate that as close as 3 ft to a pile and within the pile group, this increase in resistance is not apparent and the patern of limit of compaction must differ radically from that which the author's theories indicate. However, in the zone covered by depths of 30 ft to 38 ft, ratios of increase in resistance equal to or greater than those to be expected are found. As this zone of high compaction appears to be greater than would be theoretically expected, the writer would appreciate the author's comments.

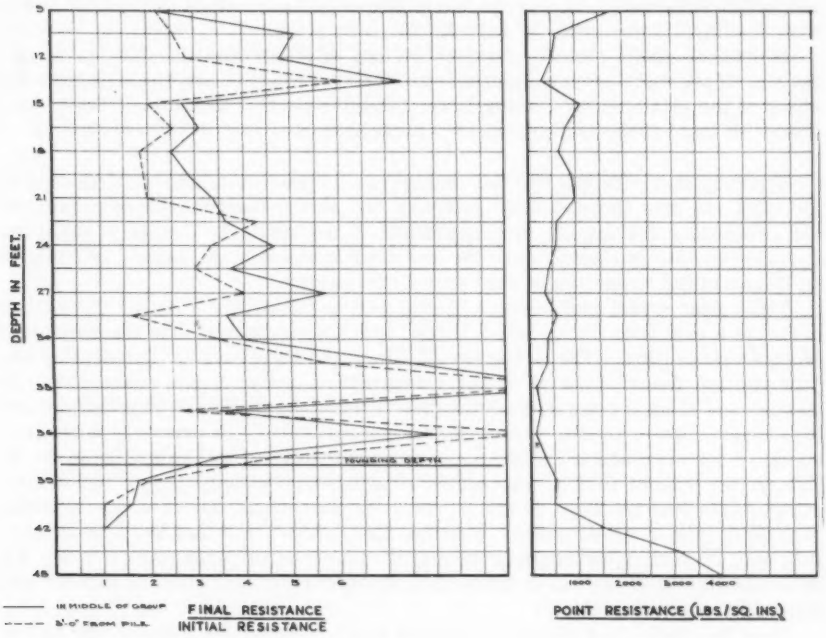

FIG. 2.—DEEP SOUNDING RESULTS.

Considering now the results for the final resistance at 3 ft from a pile, it could be expected that the ratio of increase would be of the order of 1.8. It is noted that this ratio is generally obtained down to about 22 ft but is considerably exceeded between 22 ft and 30 ft. It will also be noted that where the ratio is generally similar, the initial resistances are of the same order as those reported by the author, and it is only when low initial resistances, indicative of very loose sands, are found that the ratios are much higher. The impression gained, therefore, is that the driving of the piles tend toward achieving a uniform compaction, rather than a uniform increase in compaction, almost as if the displaced grains of sands were taking the path of least resistance. The Author's views on this aspect would also be appreciated.

A. A. EREMIN,[4] M. ASCE.—In this paper the author has shown an interesting method of estimating the bearing loading on piles, group of piles, and caissons in the cohesionless soil. Furthermore, Mr. Meyerhof considered compaction of the soil by driving the piles and has computed the settlement of pile foundation in cohesionless soil.

In practice, however, it is often difficult to pass the line between cohesive and cohesionless soils as far as pile driving and compaction are concerned. This is especially true in the moist, silty, or fine sand soils. Likewise, physical and elastic properties of soils often varie within short distances. Also, the

[4] Assoc. Bridge Engr., State Highways, Bridge Dept. Sacramento, Calif.

amount of soil compaction in the group pile foundation varies in the wide limits with the method and order of driving the outer piles.

It shall be noted, however, that the method of pile foundation study developed by Mr. Meyerhof for cohesionless soils may be used as a limiting base for study of the pile bearing loading in the cohesive soils. Therefore, the author deserves high credit for his valuable contribution.

YOSHICHIKA NISHIDA.[5]—The author has presented a fine contribution to the study of pile foundations. Although the author used the "peak pressure" for estimating the degree of compaction, it seems to be better to adopt the compaction energy itself, since the dry density of soils, in Japan, is found to be linear to the logarithm of the compaction energy.

The writer would like to learn how to derive the author's Eq. 1. The author presents a new term "compaction index" which must be useful in the treatment of sand compaction. The writer would like to do recalculation by adopting the "compaction index" instead of the compression index in his analyses of the mechanism of sand compaction by a pile in which he made use of the compression index. However, the writer finds some difficulties in practical measurements of the compaction index, though it has been defined by the author in Eq. 4. The writer cannot find a reasonable relationship between Eq. 3 and 4, and particularly between Eqs. 3 and 6. May he understand that it is due to some analogous type of relationship? May he, also, understand that Eq. 6 is obtainable only by putting the relative density D instead of the void ratio 2 in Eq. 4? He would like to know how to measure in practice the void ratio in loosest and in densest state of packing.

Does the coefficient of earth pressure in Eq. 10 mean the passive one or the one at rest?

In order to give the relative density D_r of the material at the particular point the author makes use of only the major principal stress, but the writer thinks that the density D_r, corresponding to the void ratio e, should be dependent upon the sum of three principal stresses at the point and not only upon the major one. It is particularly noted in the elastic zone. In the writer's computations on the compaction zone around a pile shaft, after solving the equilibrium equations of stress under some boundary conditions, some interesting data was evolved. Since it was done while ignoring the influence of compaction on the internal friction angle ϕ, the author's comments on the process of iteration on the ϕ value of Eq. 12 should be applied to the writer's analyses so as to make it more complete.

The major principal stress ratio near the pile shaft seems to be too great, even if ϕ becomes 35° from its original value 30°, although the rapid increase towards the shaft can be reasonable. The writer thinks that Eq. 10 is quite reasonable for the plastic zone near the pile base but it must be applied under some corrections for the soil around the pile shaft. At the boundary between two zones of plastic and elastic around the pile base the pressure p_1 obtained by Eq. 10 should be equal to that obtained from Eq. 11. Is it so in Fig. 3?

The author's opinion, that the limits of compacted soil zone are at points where the major principal stress ratio is equal to 3 for $\phi = 30°$, is based on the earth pressure theory in the two dimensional problems. No information has been given to the three dimensional earth pressure that is applied for the pile problems. The author states that the compacted zone along the shaft has an overall width of about 6 times the shaft diameter and the p_1 at the outer limits

[5] Dr. Engr., Lecturer at Kanazawa Univ. Kanazawa-shi, Japan.

of the zone of soil compaction is given by Eq. 13. The writer agrees with the estimate of "about 6 times" itself, but he does not understand how to derive Eq. 13. If the soil around the pile shaft is compacted outward, horizontally, the writer can show the stresses in the soil, under the condition that the vertical normal stress $\sigma_z = \gamma z$, as follows:

In the plastic zone of soil in failure near around the pile shaft:

$$\sigma_r = K_o \gamma z (1 + \sin \phi) \left(\frac{R}{r}\right)^{\frac{2 \sin \phi}{1 + \sin \phi}} \quad \ldots\ldots\ldots\ldots (1)$$

$$\sigma_t = K_o \gamma z (1 - \sin \phi) \left(\frac{R}{r}\right)^{\frac{2 \sin \phi}{1 + \sin \phi}} \quad \ldots\ldots\ldots\ldots (2)$$

In the elastic zone outside of the above plastic zone:

$$\sigma_r = K_o \gamma z \left(1 + \frac{R^2}{r^2} \sin \phi\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

$$\sigma_t = K_o \gamma z \left(1 = \frac{R^2}{r^2} \sin \phi\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots (4)$$

in which σ_r, σ_t are the radial and tangential normal stresses, respectively, R is the radius of the compacted soil zone, r is the radial distance from the pile axis, and z is the depth from the ground surface. These four equations fulfill the condition of strict equilibrium. The R can be obtained from

$$\frac{a^2 \left(1 + e_o\right)}{c_c} = \int_a^R 2 r \log_{10} \frac{1 + 2 K_o \left(\frac{R}{r}\right)^{\frac{2 \sin \phi}{1 + \sin \phi}}}{1 + 2 K_o} dr \quad \ldots\ldots (5)$$

in which a is the pile radius.

The radial stress increases rapidly towards the shaft and is maximum at r = a, as shown in Eq. 1. The maximum stress of horizontal does not exceed the passive earth pressure near the ground surface. Then,

$$K_o \gamma z (1 + \sin \phi) \left(\frac{R}{a}\right)^{\frac{2 \sin \phi}{1 + \sin \phi}} < = \frac{1 + \sin \phi}{1 = \sin \phi} \gamma z$$

$$\left(\frac{R}{a}\right)^{\frac{2 \sin \phi}{1 + \sin \phi}} < = K_o \frac{1}{(1 = \sin \phi)}$$

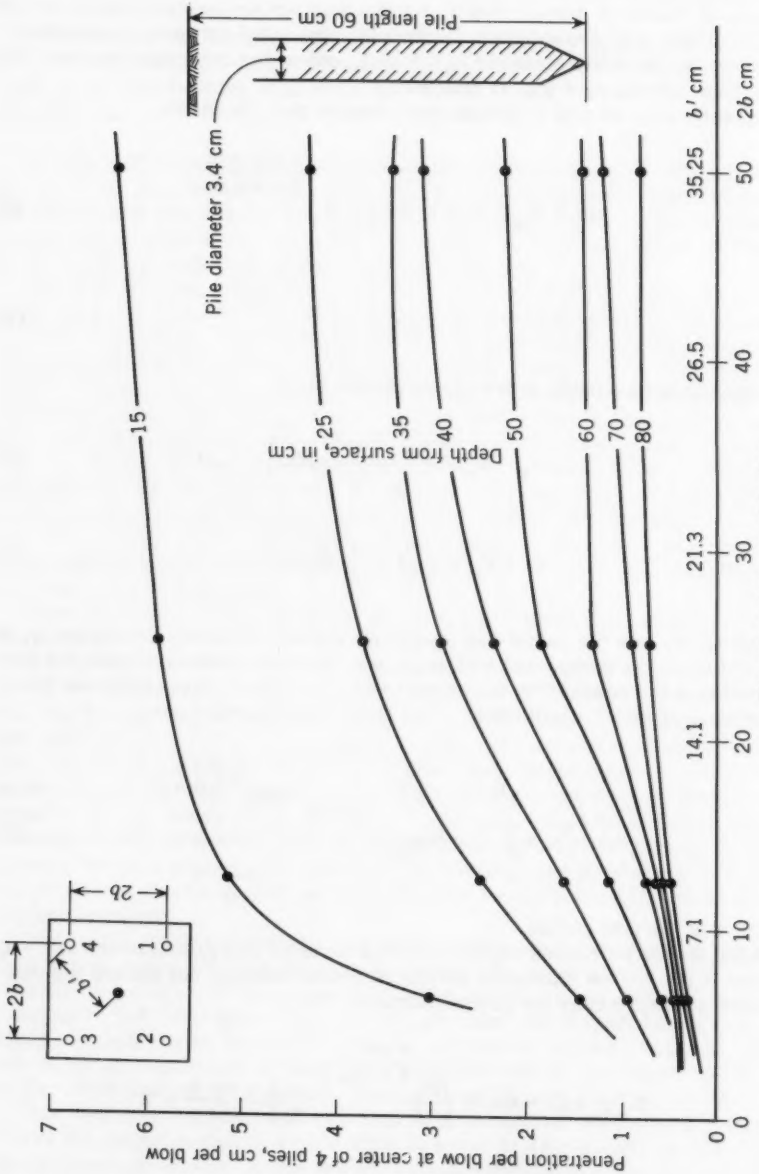

FIG. 1.—PENETRATION RESISTANCE AROUND A DRIVEN PILE

if

$$K_o = 0.5 \quad \zeta = 30°; \quad R = 8a$$

$$K_o = 0.7 \quad \zeta = 40°; \quad R = 5.9a$$

A similar solution for the pile base is yet unknown and it is being computed by the writer's. The author's comments, that the depth of the compacted zone around the pile base is up to 4 times the pile diameter, is very useful to the writer's study that was previously mentioned. In Fig. 3, the limits of the compacted soil zone are determined by the major principal stress ratio being 3, and in Fig. 5, it is done by being one. The difference between the two methods should be explained.

The writer presents one of his experiments in Fig. 1, where the penetration resistance around a driven pile was compared for the sand of the original $\phi = 35°$. The penetrations were done, at the center of 4 piles of the same dimensions, on a smooth steel bar (weight of 432 grams, diameter of 7 mm; length of 130 cm) driven by the shock due to the 20 cm free fall of the 150 gram weight. The observations indicate the limits of the compacted zone, found by the change of the penetration resistance as 8 to 12 times the shaft diameter, where 4 piles made it more remarkable than by one pile.

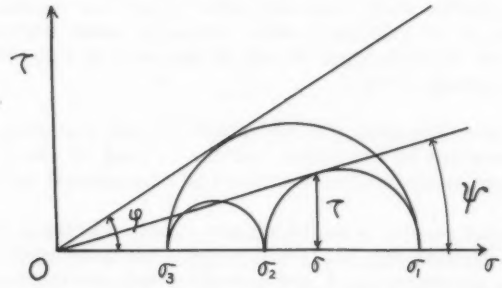

FIG. 2.—MOHR'S CIRCLE

The author says that the 60% to 100% increase of bearing capacity of a driven pile is due to an increase of internal friction angle due to the soil compaction. The writer agrees in principle with this comment and it must be particularly so at the pile base. According to his study, the skin friction can increase greatly due to the outward prestressing of the soil by a driven pile even under the constant internal friction angle, the same as the original value, as easily seen by putting r = a in Eq. 1. The base resistance should be considered in the three dimensional problems, and the internal friction angle must, in fact, be different from ϕ itself.

As one of its solution the writer shows the Mohr's circle, of three dimensions, on principal stresses. (See Fig. 2) The minimum principal stress acts normal to the vertical plane, and, accordingly the plastic zone, can be analysed by using instead of ϕ, if the soil failure occurs under the condition of $\sigma_1 = \sigma_3 \times [x (1 + \sin \phi)/(1 - \sin \phi)]$.

When this theory is applied, the theoretical point resistance would be 25% greater instead of 60% for a driven pile in the loose sand ($\phi = 30°$) with a 5° increase of internal friction at the pile base. The compaction by driving a pile has more influence on the soil around the pile shaft than the soil below the pile base, according to the writer's experience. He can show an example of calculations. The group piles, closely spaced—less than about 3 to 4 times the pile diameter—behave as one block foundation. The author's comments, based on the model test, that for the loose sands the bearing capacity of driven piles is nearly twice that of buried piles, agrees closely with the results of the writer's computations.

Some analyses have been done concerning the results of loading tests of driven piles. In the practical point of view it can be considered that the skin friction of piles is a linear distribution with the depth, but it must be noted that the skin friction does not distribute in linear to the depth according to measurements in experiments or in the fields. Accordingly, the earth pressure coefficient can not easily be available at the lateral surface of the pile in the strict meaning, except in the case where it is used as the average value through the full length of the pile. The writer has already made some comments concerning this problem.[6]

The author's idea, that the angle of internal friction ϕ can be deduced from the analysis of the energy per blow required for driving a pile and for installing the cassion or for expanding the base, has merit. The writer would like to develop this idea in his study, with the author's advice, because he has measured the stress at the pile base while driving a model pile into sands. He wishes to know how to obtain Eqs. 25 and 26, since he is also interested in the settlement of pile foundations.

W. I. LOW.[7]—Mr. Meyerhof has presented a most comprehensive and interesting paper in which he has shown that the factors of safety and loadings commonly used are within the limits dictated by a theoretical analysis of group settlement.

The experimental results of model impact compaction tests are of interest to the writer, as similar full-scale compaction has been used in the Durban, South Africa area to consolidate a site underlain by loose saturated silty sand, on which large oil storage tanks have since been erected. A 10,000 lb concrete block was repeatedly dropped from a height of approximately 6 ft, and the effect was gaged by carrying out static cone penetration tests before and after compaction. A typical pair of results of soundings carried out at the same position is given in Table 1. It was found that a depth of approximately 15 ft of soil showed an appreciable increase in penetration resistance. Where the original cone resistance was of the order of 10 tons per sq ft (150 psi) or more, an increase of two to three times was apparent, but where the initial resistance was very low, ratios of final to initial resistance between 30 and 50 were found.

In the estuarine bay area of Durban soils generally exhibit considerably lower penetration resistances than those quoted by the author for loose sand. It is, in fact, not uncommon to find zones of fine sands of such low strength that no pressure is recorded in the hydraulic cylinder of the penetrometer when the cone is advanced. As, however, the cone does not sink under the weight of the operating rods, it could be assumed that this weight in fact represents the penetration resistance of the soil at that level. Such soils also

[6] Transactions, ASCE, Vol. 123, p. 744.
[7] Partner, Kantey, von Geusau and Partners, Cons. Engineers, Durban, South Africa.

showed very high ratios of final to initial penetration resistance when tests were carried out before and after pile driving.

Table 2 shows the results of static cone penetration tests at a site underlain by saturated fine sands, carried out before piling was started, and in the center of several groups of four shell piles. These piles could, for the purpose of the present comparison, be taken to correspond to precast driven piles. The positions of the soundings and pile layout are shown on Fig. 1. The distance from the initial to the final test, the direction of which is indicated by

TABLE 1

Depth Feet	Initial Cone Resistance, psi	Final Cone Resistance after Impact Compaction, psi	Ratio Final/Initial
1.5	200	110	0.5
3.0	400	425	1.1
4.5	150	500	3.3
6.0	150	300	2.0
7.5	8	250	31.0
9.0	10	500	50.0
10.5	150	500	3.3
12.0	150	250	1.7
13.5	14	250	18.0
15.0	150	350	2.3
16.5	250	350	1.4
18.0	250	400	1.6
19.5	250	350	1.4
21.0	250	350	1.4
Tests continued to 50 ft			

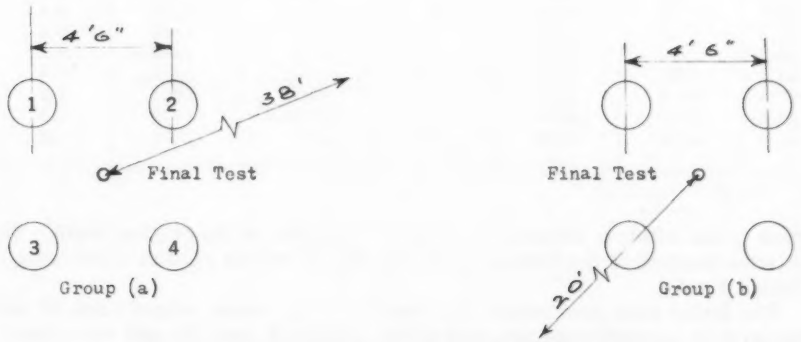

FIG. 1.—CONE PENETRATION TESTS-DRIVEN PILES

the arrow, is 38 ft for group a and 30 ft for group b. The lengths of the piles are 18 ft, 16 ft, 20 ft, 18 ft, 12.5 ft, 15 ft, 13.5 ft and 12 ft for piles 1 through 8, respectively.

The ratios of final to initial cone resistance measured at this site are again higher than those measured by the author, reaching values of 189 to 190 in some cases. This would seem to confirm the author's findings regarding the parabolic relationship between penetration resistance and relative density in sands.

At a distance of 1.75 D from the pile shafts, and at the level of the pile tip, ratios of 2.7 to 14.1 were found, the final value being, to a large extent, dependent on the original penetration resistance of the soil. In this investigation a distance of some 20 ft to 40 ft separated the final and initial tests, so some natural variation in test results could be expected, which might account for some of the variation in resistance ratios.

Recently, however, through the courtesy of Frankipile South Africa (Pty.) Ltd., the writer has been able to analyse a series of tests carried out at a site in the central Durban area. This site is underlain by saturated silty sand, with two layers of sandy clay from a 10 ft to 14 ft depth and from a 26 ft to 27 ft depth. Eight cone penetration tests were carried out initially, and after the placing of groups of 18-in.-diam Franki displacement caissons, further tests

TABLE 2

	Group (a)			Group (b)		
Depth Feet	Initial Resistance, psi	Final Resistance, psi	Ratio Final/ Initial	Initial Resistance, psi	Final Resistance, psi	Ratio Final/ Initial Resistance
1.5	380	80	0.2	370	3	0.
3.0	190	290	1.5	380	280	0.7
4.5	4	680	170.	760	470	0.6
6.0	190	1320	7.0	460	370	0.8
7.5	760	1650	2.2	530	450	0.8
9.0	190	2000	10.5	225	670	3.0
10.5	11	2060	188.	490	1000	2.0
12.0	760	2000	2.6	340	1000	3.0
13.5	190	2000	10.5	660	2840	4.3
15.0	380	2060	5.5	660	2700	4.1
16.5	380	2460	6.5	750	2050	2.7
18.0	570	2670	4.7	1020	1500	1.5
19.5	190	2670	14.1	1100	1040	1.0
21.0	22	2080	95.	1120	1320	1.2
22.5	95	1750	18.4	3020	1500	0.5
24.0	795	1170	1.5	2640	1500	0.6
25.5	1135	1750	1.5	3000	2820	0.9
27.0	1510	2260	1.4	2240	2620	1.2
28.5	1890	2540	1.3	1520	2840	1.9
30.0	3010	3000	1.0	2860	2860	1.0

were made within a distance of 2 ft to 4 ft of two of the original tests. The relative position of the tests is shown on Fig. 2, and the results of the tests in Tables 3 and 4.

The initial cone penetration resistances in the sands ranged from 35 tons per sq ft to over 300 tons per sq ft (500 - 4500 psi), and the soil was characterized by alternate shallow layers of dense and loose sand. The penetration results indicated that a sufficiently high cone resistance was consistently found below a depth of 38 ft to 42 ft across the site, and bases of the Franki caissons were formed at 36 ft to 38 ft on the assumption that the energy put into forming the base would adequately compact the underling 2 ft to 4 ft of weaker soil.

This assumption was confirmed by subsequent penetration tests at distances of $1\frac{1}{2}$- and $2\frac{1}{2}$-diam from the shaft. The ratio of final to initial penetration resistance in the weaker zones varied from 11 just above base level to 3.7 at

a level 1-diameter below the base, while in the more dense zones ratios of 1 to 2 were found close to the base level. In the areas above the base and alongside the shaft, the ratios were somewhat less than would be expected from the author's figures. For instance, in the initial test No. 2 and subsequent test No. 2A, only 2 ft apart, that are shown graphically on Fig. 3, the ratios alongside the shaft were often less than 1, whereas a figure of 1.7 to 2 could be expected. Both these tests were carried some 20 ft below the compacted zones, as were the tests at the site where impact compaction was used. It is interesting to note that the natural variation in soil strength over a distance of 2 ft

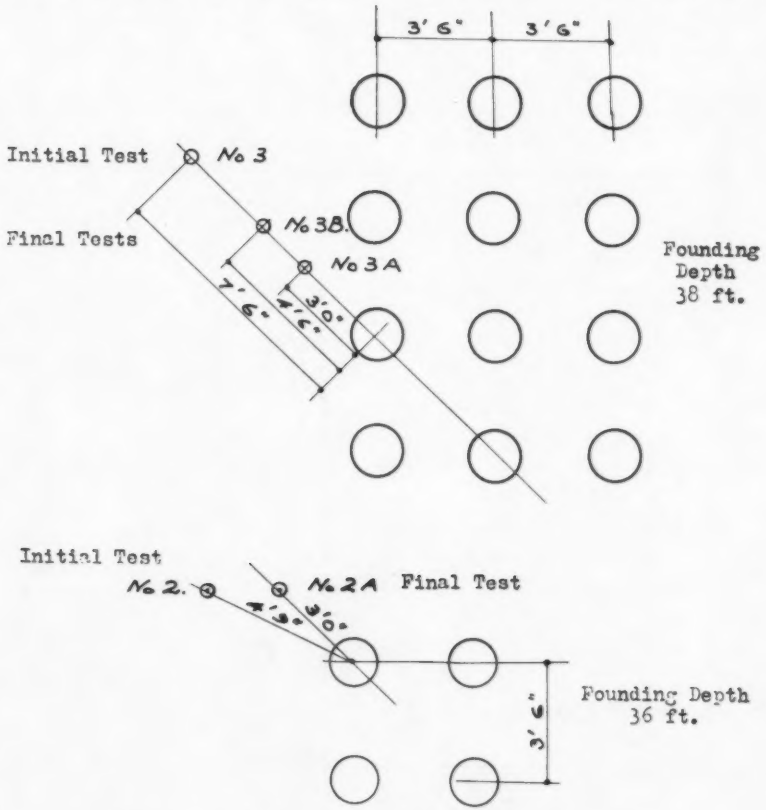

FIG. 2.—CONE PENETRATION TESTS-FRANKI CAISSONS

or less gave ratios of final to initial penetration resistance varying from 0.6 to 2.5, though the expected ratio of 1 in the undisturbed soil was generally found.

Considering the results of the tests at both piled sites, the increase in penetration resistance was found to extend to a depth not exceeding $2\frac{1}{2}$ diameters below the base, whereas, from the author's figures, an increase could be expected to a depth of 4 to 5 diam below a driven pile, and 5 to 6 diam below a Franki caisson.

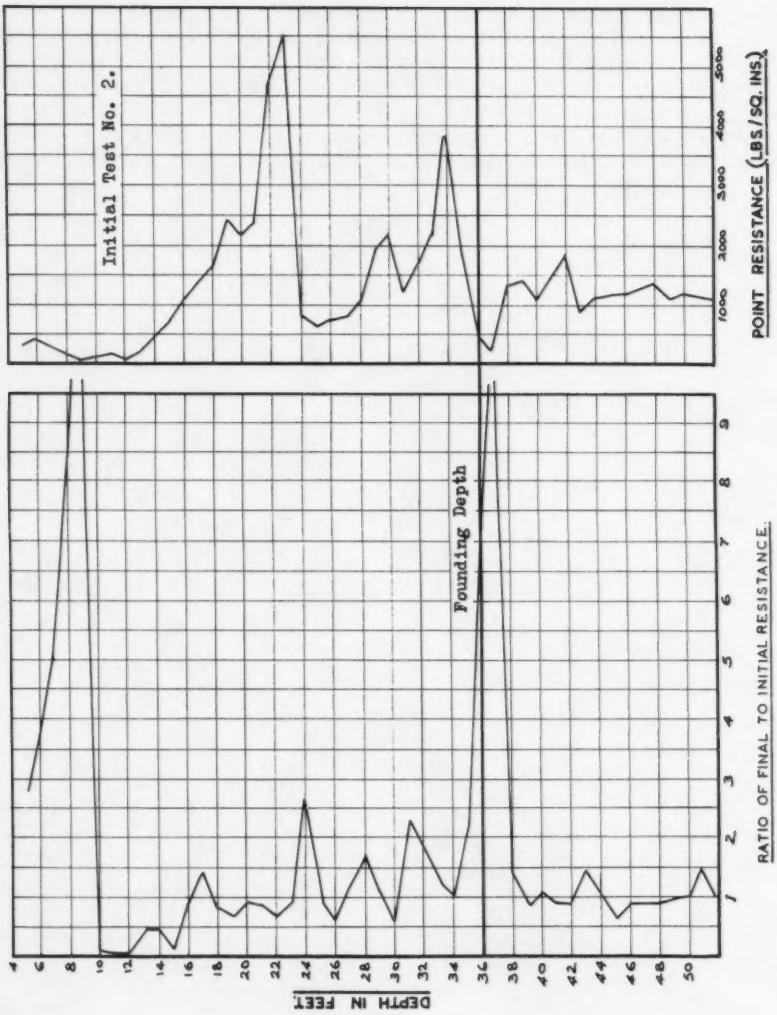

FIG. 3.—CONE PENETRATION TEST RESULTS

In view of these variations between results obtained in very loose South African soil and those quoted by the author for American and Canadian soils, it would be of interest to have the author's comments on whether he considered that this variation was due largely to differing initial relative densities or to

TABLE 3.—TESTS NOS. 2 AND 2A. NEAR FRANKI CAISSONS

Depth Feet	2 1/2D from Shaft Initial Resistance, psi	1 1/2D from Shaft	
		Final Resistance, psi	Ratio Final/Initial
5	375	1040	2.8
6	430	1570	3.7
7	350	1810	5.2
8	180	1460	8.1
9	10	140	14.0
10	75	10	0.13
11	200	11	0.06
12	100	12	0.12
13	200	105	0.5
14	500	280	0.5
15	750	140	0.19
16	1150	900	0.8
17	1400	2020	1.5
18	1675	1290	0.8
19	2450	1810	0.7
20	2250	1950	0.9
21	2450	2020	0.8
22	4600	3300	0.7
23	5550	5200	0.9
24	850	2600	3.1
25	700	610	0.9
26	780	435	0.6
27	850	1050	1.2
28	1020	1750	1.7
29	1980	2260	1.1
30	2350	1300	0.6
31	1300	2960	2.3
32	1580	2960	1.9
33	2250	2780	1.2
34	3800	3820	1.0
35	1850	4000	2.2
36	500	3480	7.0
37	250	2780	11.1
38	1375	1910	1.4
39	1450	1130	0.8
40	1160	1300	1.1
41	1400	1300	0.9
42	1800	1650	0.9
43	900	1390	1.5
44	1120	1390	1.2

Tests were continued to 60 ft.

the difference in the use of static and dynamic penetration tests. The writer's experience in comparing the pattern of static and dynamic cone penetration tests was that the dynamic test tended to give a smoother curve, with a levelling out of both the very high and very low points found in static tests. This in turn would give less variation in the ratios of final to initial penetration resistance.

The author's curves for settlements of pile groups are a valuable extension of the previous work of Skempton, Yassin and Gibson. However, the practical and economical use of these curves is essentially dependent on adequate load tests on individual piles. In South Africa, when load testing has been carried

TABLE 4.—TESTS NOS. 3, 3A AND 3B NEAR FRANKI CAISSONS

Depth Feet	4 1/2D from Shaft	1 1/2D from Shaft		2 1/2D from Shaft	
	Initial Resistance, psi	Final Resistance, psi	Ratio	Final Resistance, psi	Ratio
5	275	1180	4.3	975	3.5
6	350	1115	3.2	1460	4.2
7	400	450	1.1	700	1.7
8	200	8	0.05	70	0.3
9	10	10	1.0	10	1.0
10	100	10	0.1	70	0.7
11	100	12	0.1	12	0.1
12	100	209	2.1	12	0.1
13	100	245	2.5	70	0.7
14	160	900	5.6	450	2.8
15	600	1250	2.1	870	1.4
16	1150	1180	1.6	1150	1.3
17	1500	2090	1.4	1460	1.0
18	2000	2870	1.4	1390	0.7
19	1920	2870	1.5	1250	0.7
20	1780	4000	2.2	1850	1.0
21	1980	4600	2.3	2600	1.3
22	2300	6000	2.6	4180	1.8
23	3750	6100	1.6	3820	1.0
24	4700	4350	0.9	1390	0.3
25	1050	740	0.7	280	0.3
26	400	28	0.07	490	1.2
27	350	420	1.2	835	2.4
28	450	380	0.8	2600	5.8
29	1200	2260	1.9	2260	1.9
30	1750	1830	1.0	2440	1.4
31	1950	3480	1.8	3130	1.6
32	1750	4450	2.5	3130	1.8
33	1950	6200	3.2	4180	2.1
34	2800	7150	2.5	3480	1.2
34.5	2800	8700	3.1	–	–
35	2950	–	–	3480	1.2
36	3100	–	–	4000	1.3
37	250	–	–	2780	11.1
38	600	–	–	3020	5.0
39	1150	–	–	1910	1.7
40	500	–	–	1830	3.7
41	1450	–	–	1910	1.3
42	1600	–	–	2080	1.3

out it has usually been limited for reasons of cost to loads of 150% to 200% of the design load of the pile. Settlements under such loads are usually too small to use the author's method of extrapolation to determine the ultimate load and, hence, the actual factor of safety of the group.

A REVIEW AND EVALUATION OF SOIL-CEMENT PAVEMENTS[a]

Discussion by Miles D. Catton, D. J. Maclean and K. E. Clare

MILES D. CATTON,[1] F. ASCE.—The authors have presented a thorough review of the literature pertaining to the properties of soil-cement and the critical examination and analysis of airfield performance data. The paper adds materially to the knowledge of this construction material.

The inherent simplicity of construction is demonstrated by the performance record of the United States airfield projects reviewed. These projects were built by inexperienced crews under time limit pressure. They are included in a total of over 22,000,000 sq yds of soil-cement airport paving built during 1941-1945, under wartime emergency conditions.[2] One Navy airfield project totaled 1,685,665 sq yds. Also, military installations included substantial yardages of soil-cement roads and streets. Many unusual construction records were made.[3] This work has a record of good-to-excellent performance.

The discussion includes uses of "cement modified soil." Early road projects built by the highway departments of Kansas[4] and Oklahoma[5] defined and demonstrated some of the factors governing soil-cement mixtures of this category. Knowledge of these factors possessed by the United States Corps of Engineers, Dept. of the Army was used by them to solve a critical staging area problem encountered at Omaha Beach during the Normandy (France) invasion.[6]

Another area of discussion includes valuable information on some of the physico-chemical factors involved in soil-cement technology. The earliest pioneering investigation of these factors was undertaken by H. F. Winterkorn, at the University of Missouri.[7] Also, the pioneering work of the North Carolina State Highway Commission correlating pedology and soil-cement was an outstanding contribution to soil-cement technology.[8]

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[a] December, 1959, by James K. Mitchell and Dean R. Freitag.

[1] Asst. to Vice Pres. for Research and Development, Portland Cement Assn., Skokie, Ill.

[2] "Summaries of Soil-Cement Construction, SC-104," Portland Cement Assn., Chicago, Ill.

[3] "Some Wartime Soil-Cement Experiences," by Miles D. Catton, Proceedings, 24th Annual Meeting, Highway Research Bd., Vol. 24, 1944.

[4] "Laboratory Investigations of Soil-Cement Mixtures for Subgrade Treatment in Kansas," by Miles D. Catton, Proceedings, 17th Annual Meeting, Part II, Highway Research Bd., 1937.

[5] "Concrete Pavement Subgrade Design, Construction, Control," by C. R. Reid, Proceedings, 19th Annual Meeting, Highway Research Bd., 1939.

[6] "Obstables Unawares," by Col. Leland B. Kuhre, C. E., Military Engineer, May, June, 1949.

[7] "Surface Chemical Factors of Importance in the Hardening of Soils by Means of Portland Cement," by H. F. Winterkorn, H. J. Gibbs, and R. E. Fohrman, Proceedings, 22nd Annual Meeting, Highway Research Bd., 1942.

[8] "Sampling, Soil Classification and Cement Requirement," by L. D. Hicks, Proceedings, 14th Annual Meeting, Highway Research Bd., 1939.

The discussion of mix design (cement content) includes reference to the American Society of Testing Materials (ASTM) soil-cement test procedures and criteria governing cement requirements for paving mixtures. Mention was made of two criteria of basic importance; 1) moisture content at any time during test shall not exceed total voids at time of compaction, and 2) volume change during test shall not be greater than 2%. Years of experience with soils occurring in the United States has demonstrated that these two criteria seldom govern and, hence, do not receive particular emphasis at the present time. However, an occasional soil does possess extremely high volume change characteristics that accompany moisture changes. The two criteria are an intimate part of the involved and interrelated reactions of soil and cement.

The aforementioned criteria, as well as the others developed for use with the ASTM soil-cement wet-dry and freeze-thaw tests, are based inherently on some of the physico-chemical properties of soil and the chemical properties of cement. While the literature sometimes refers to the tests as durability tests, they do not represent or simulate any particular climate or outdoor exposure (and, to simulate a particular climate or outdoor exposure in the laboratory would require the design and construction of most complicated and expensive electronic control devices). Instead, they are designed to determine the quantities of cement required to hold the compacted mass of soil-cement together to produce a structural material.[9] The wet-dry test simulates conditions to evaluate some of the inherent shrinkage properties of fine grain soils. The freeze-thaw test simulates conditions that allow an evaluation of some of the expansive properties of fine grain soils. The maximum temperature in the wet-dry test is below that which will produce physical changes in the soil. While the wet-dry test also evaluates some of the expansive forces present, it is not sufficient unto itself because the higher temperatures are not present on construction. They may accelerate cement hydration and induce physico-chemical changes in the mixture which will not occur on field installations. On the other hand, these higher temperatures are not encountered (purposely) in the freeze-thaw test. To summarize—as basic considerations the wet-dry and freeze-thaw tests were designed:

1) to simulate the tensile and compressive forces generated internally in compacted soil-cement as a result of moisture changes; and

2) to evaluate in a practical way the influence of; a) the chemistry of cement, b) the surface chemistry of soils, and c) the physico-chemical properties of soils on soil and cement reactions.

As pointed out by the authors, a compressive strength criteria of itself does not necessarily determine minimum cement contents for paving work. However, once a soil has been identified and cement requirements have been determined by the wet-dry test and the freeze-thaw test and their accompanying criteria (or by procedures based on correlations with the wet-dry and freeze-thaw tests), the compressive strength of that suitable, compacted mixture may be used as a minimum criteria for similar soils.

The structural design requirements for soil-cement paving are ably reviewed and discussed by the authors. Two design variables, thickness and compressive and flexural strength (obtained by increasing cement contents), are particularly important.

The following comments are based on the minimum cement content required to meet the mixture design criteria used in conjunction with the soil-cement

9 "Early Soil-Cement Research and Development," by Miles D. Catton, Proceedings, ASCE, Vol: 85, No. HW 1, January, 1959.

wet-dry and freeze-thaw tests. One of the most valuable mechanical properties of soil-cement to be considered in soil-cement pavement design is its modulus of elasticity. In general, and in comparison with concrete, this modulus is low. This low value is of major importance since major deflections may take place under loading without rupture of the soil-cement. These large deflections result in increases in the area of soil-cement and the area of subgrade support utilized in carrying the imposed load. It is the judgment of the writer, based on extensive experience on the Bates (Illinois) Test Road and on observations of soil-cement during construction, and, later, of projects in service, that the relatively low modulus of elasticity prevailing on soil-cement paving (that permits relatively high deflections without rupture) is the predominant mechanical property contributing to its high load carrying capacity.

In further consideration of structural design of soil-cement paving, a 6-in. depth of compacted soil-cement, with the minimum cement content specified previously, will be used as a "bench mark." The vehicle load carrying capacity of this unit will be governed by the mechanical properties of the soil-cement and of the underlying subgrade. Two procedures may be followed or combined to increase load carrying capacity. Cement contents of the mixture may be increased to produce greater mechanical strength (giving due recognition to modulus of elasticity) and the depth of paving may be increased. Whether these two design variables are used independently or in various combinations, the basic objective is to increase the area of load distribution on the subgrade so that the maximum transmitted pressure at any point does not rupture (fail) the subgrade.

The rate of strength gain of soil-cement paving as influenced by construction temperatures is cited. In general, as is to be expected, the strength gain is similar to concrete. However, in concrete paving design, a certain governing strength is selected, based on normal construction season conditions. When low strengths prevail, resulting from low temperatures and slow cement hydration, permissible traffic on the pavement is governed by the prevailing strength of the concrete. A similar approach and criteria would appear tenable for soil-cement paving. In other words, specify cement contents for soil-cement on the basis of normal construction temperatures. In addition, experience has repeatedly demonstrated (particularly during the emergency construction period 1941-1945) that the loads imposed by compaction equipment exceed subsequent loads. This compaction equipment must be carried without rupture of the subgrade or paving or else adequate compaction is not attained. Thus, the paving is strong enough for all but exceptionally heavy wheel loads as soon as compacted and before cement hydration proceeds.

As cement hydration proceeds, the load carrying capacity of the paving is increased materially. This increased strength is utilized later, should the subgrade be weakened by increases in moisture content. The writer has supervised construction on road projects built during late fall, in cool to freezing weather, with traffic imposed upon completion, and structural failures were not induced. There are records of many similar experiences.

D. J. MACLEAN[10] and K. E. CLARE.[11]—Messrs. Mitchell and Freitag have produced a comprehensive review of the design and performance of soil cement pavements. In this review a number of references are made to research work carried out at the Road Research Laboratory in Great Britain,

[10] Head, Soil Sect., Road Research Lab., Great Britain.
[11] Tropical Sect., Road Research Lab., Great Britain.

and it may be of interest to comment on some of these references with a view to expanding or clarifying the information given by the authors.

In Great Britain, it is the presence of organic matter in the surface layers of soils that proves to be the greatest hindrance to the use of soil-cement, particularly where construction by the mix-in-place method is contemplated. Our laboritory's investigations have shown that the principal advantage of the pedological classification of soil profiles is to enable a fairly reliable estimate to be made of the depth at which soil occurs sufficiently free from organic matter to be suitable for stabilization with Portland cement. There are only five types of profile that have to be considered in this connection in Great Britain. They are (1) the calcareous, (2) the high base status brown earth profiles, for which no difficulty arises from the presence of organic matter right up to the surface of the ground, (3) gley profiles, for which soils are suitable for stabilization with ordinary Portland cement below a depth of 15 in., (4) the low base status brown earths, and (5) the pdozol profiles, for which soils are suitable for stabilization at depths exceeding from 2 to 5 ft.

An analysis of the relation, for organic soils taken from these profiles, between their suitability for stabilization and their pH confirmed that, for conditions in Great Britain, alkaline soils are more suitable for stabilization than acidic soils. Whereas practically all the alkaline soils reacted favorably with Portland cement, acidic soils with an organic content greater than 1% were unsuitable and those with an organic content less than 1% were sometimes suitable and sometimes unsuitable.

The acidity of soils in Great Britain usually arises from the presence of humic acids due to the organic matter. In tropical countries, where acidic conditions do not result from the presence of organic matter in the soils, no difficulty is found in stabilizing acidic soils with Portland cement. This is possibly due to the free iron present, that would be expected to combine with the humic acids and inactivate them.

A detailed investigation has recently been made of the value of calcium chloride for increasing the hardening obtained when organic soils are treated with Portland cement. This showed that calcium chloride was ineffective with soils containing highly active organic matter and was only effective in what we have termed a transition zone that occurs in soil profiles between an upper zone that contains highly active organic matter and a lower zone that is relatively free from organic matter.

While the higher early strength obtained with some cement-stabilized soils by using a small proportion of sodium sulphate as a secondary additive with the Portland cement are of undoubted interest, one feels that long-term durability tests are required to establish beyond doubt the value of this treatment.

There is a necessity for further research to determine the conditions under which soils with expanding-lattice-type minerals can be successfully stabilized with Portland cement. The results of limited tests that have been made at the Road Research Laboratory indicate that with such soils the soil-cement mixture should be compacted and cured at a moisture content sufficiently high to prevent the hardened material swelling under the prevailing moisture conditions in the pavement. Even so, more knowledge is required of the behaviour of such materials under fluctuating moisture conditions before they can be considered for use in practice.

With regard to the effect of temperature on the strength of soil-cement, a recent investigation at the Road Research Laboratory has shown that the increase in strength per degree rise in temperature is greater at high than at

low ambient temperatures for stabilized cohesive soils but is little affected by the ambient temperature for stabilized granular soils. This difference is attributed to an acceleration for cohesive soils of a pozzolanic action between clay particles and lime released by the hydrolysis of the Portland cement.

We are sorry to note that the authors have been misinformed about the basis for determining cement contents in Great Britain. In 1939, the Road Research Laboratory correlated the results of compressive strength tests on soil-cement cubes with their ability to satisfy the requirements of the American wet-dry and freeze-thaw tests. This work showed that soil-cement with a minimum cube strength of 250 psi, at 7 days, generally, satisfied the requirements of the American tests. From 1945 onwards, the laboratory determined cement contents required to stabilize soils adequately on the basis of a compressive strength, obtained on cylindrical specimens prepared in the laboratory, of 250 psi at 7 days.

A considerable mileage of lightly-trafficked roads was constructed with soil-cement bases conforming to this requirement, and these roads, which were usually surfaced with a double surface-dressing, have generally given satisfactory service. With the comparatively inefficient mix-in-place form of construction used in the period 1945-1950, field strengths of the order of only 60% of the laboratory strengths were obtained, and this means that soil-cement bases with a field strength of only about 150 psi at 7 days were satisfactory for lightly-trafficked roads.

Experiments have been carried out by the Road Research Laboratory to assess the performance of soil-cement bases for roads with heavy traffic. These have shown that their performance depends, as would be expected, on the thickness and stiffness of the surfacing material, but for the two-course surfacings 4 in. thick, it appears that a field compressive strength of the order of 300 to 350 psi at 7 days is required to give a satisfactory performance.

"Blow-ups" of soil-cement pavements have been reported to occur in hot weather. Van der Sluis[12] has reported that this effect occurred on sections of soil-cement roads in Holland where the compressive strength of the soil-cement was higher than average, while Barrie and Cottington[13] reported "blow-ups" on a soil-cement runway in Great Britain when the average compressive strength of the soil-cement was 680 psi at 7 days. There is no evidence from experience of soil-cement in Great Britain to support the view that on overall economy is likely to result from the use of high-strength soil-cement.

The authors rightly refer to the importance of uniformity of mixing, and for this reason investigations have been made in Great Britain of the distribution of cement in soil-cement roads and airfields constructed by the mix-in-place process. In one investigation, segregation of cement in a vertical direction was shown to be a cause of failure.[14] Mixing cement into damp rather than dry soil may help to prevent the segregation. Thick soil-cement pavements built up of two layers by the mix-in-place process have sometimes contained a weak layer at the bottom of the upper layer due to an inadequate depth of

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12 "Construction of Soil-Cement Roads and Road Bases in Holland," by F. A. Van der Sluis, Journal of the Inst. of Highway Engrs., Vol. 5, No. 2, 1958, pp. 79-112.

13 "Construction of a Soil-Cement Runway at Christchurch," by A. O. Barrie, and J. Cottington, Proceedings, Inst. of Civ. Engrs., Vol. 6, 1957, pp. 577-594.

14 "Some Problems in Mixing Granular Materials Used in Road Construction," by K. E. Clare, R. H. H. Kirkham, and H. G. Barnes, Pub. Wks. and Munic. Services Congress and Exhibition, 1954, Final Report, London, 1955, pp. 46-77.

mixing. In two-layer work on the La Villiaze airport in Guernsey, the weak layer has been avoided by using plant-mixed soil-cement in the upper layer.[15]

Most of the construction described by the authors is located in the comparative mild climate conditions of the British Isles, and the southern section of the United States. During the 1939-45 war, the German armed forces used soil-cement at more than 130 airfields in Europe. A number of these were located in the Soviet Union or Central Europe where severe winter freezing conditions occur. One of these inspected eight years later was in fair conditions.[16] Following their war-time success with the material, the Germans continue to use soil-cement in housing estate roads and autobahn construction. Experiments by the Royal Swedish Air Force have shown that soil-cement withstands Scandinavian climatic conditions,[17] although both there and in Germany the cement contents employed are of the order of 15% to 20%, by weight, which is higher than in British or United States practice. In hotter climates, soil-cement was successfully used at several airfields in Southern Rhodesia during the war, at Kai Tak airfield in Hong Kong,[18] Ndola airfield in Northern Rhodesia, and on main roads in Brunei,[19] Ghana Kenya, Nigeria and Uganda. Properly executed it appears to have quite satisfactory durability under tropical conditions, and with the lateritic and quartzitic gravels normally used in making road bases, cement contents of the order of 4% have been found to be satisfactory.

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15 "New Runway Construction to Deal with Increasing Air Traffic, by Anon, Concrete Quarterly, No. 43, October- December, 1959, pp. 32-4.

16 "Soil Stabilization in Germany," by K. E. Clare and R. A. Foulkes, Engineering, 1954, 178 (4622), 268-73.

17 "Svenska erfarenheter av cementstabilisering," by B. Orbom, Vag-och vattenbyggaren, Stockholm, 1956, (1), pp. 12-14.

18 "The Incorporation of Decomposed Granite in the Design and Construction of Pavements in Hong Kong," by K. K. M. Henry and H. Grace, Proceedings of the 2nd Internatl. Soil Mech. Conf., Rotterdam, Vol. 4, 1948, 190.

19 "Soil-Cement Stabilized Roads in Brunei, Borneo," by G. T. Myles, Dept. of Scientific and Industrial Research, Road Research Overseas Bulletin No. 7, Harmondsworth, 1957, (Road Research Lab).

PILE CONSTRUCTION IN PERMAFROST[a]

Discussion by J. K. Sexton, Per Hall and D. F. Murphy

J. K. SEXTON.[1]—Mr. Pihlainen's paper offers a timely summary on recent experience in the erection of permanent structures on permafrost in the Canadian North. The construction of a new town for an ultimate population of 2,500 at Inuvik on the Mackenzie River delta, to which Mr. Pihlainen refers, has given rise to a confidence and a proficiency in dealing with permafrost that augers well for the development of the north country. In this case all the public and private buildings in the serviced area of the town are elevated above and rest on the tops of piles frozen securely into the permafrost. Moreover, all the conduits for the water, sewage and central heating utilities are carried in a network of utilidors which are also supported on piles well above ground. The town site with its utilities has now been in operation for over a year, with very few cases of foundation movement. Where such movement has occurred it can be traced to an infraction of the rules for piles in permafrost which Mr. Pihlainen describes.

Unlike a pile driven in normal soil, a pile frozen into permafrost is designed to resist both upward and downward forces. The adfreezing strength developed over the area of contact between pile and permanently frozen material must be sufficient, not only to resist the vertical loading of the structure but, also, to act as an anchorage against the tendency to heave that occurs each year with the re-freezing of the active layer on the surface. As a general rule, at Inuvik, the piles were frozen into the permafrost for a distance of not less than 2 times the maximum depth of the active layer. Where this rule was observed no heaving has occurred. For some of the more important structures it had been hoped that the piles could be drilled in and frozen with their butt end downward, but difficulty of drilling through gravel lenses and the need for speed in meeting the construction program restricted this type of piling to the bases for the main fuel oil tanks at the power station. Elsewhere all piles were steamed in and placed tip down.

Wherever there was a heavy concentration of piles some anxiety was felt lest the use of steam should result in excessive melting of the permafrost, and indeed there was one case in which subsequent test excavations showed some caverns of melted material to have formed. This situation was corrected in the course of a year by insulating and refrigerating the ground surface to prevent entrance of heat from the atmosphere while the thawed patches slowly lost their heat to the adjacent permafrost and refroze. The average permafrost temperature over the piling depth is about 26°F. It is believed that this method of refreezing the thawed patches of flow of heat from the latter in a more or less horizontal direction was in itself a precaution against heaving since the latter is said to occur in the direction of heat flow.

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[a] December, 1959, by J. A. Pihlainen.

[1] Chf. Civ. Engr., Montreal Engrg. Co., Ltd., Montreal, Canada.

PER HALL,[2] M. ASCE and D. F. MURPHY.[3]—The author is to be commended for presenting a comprehensive paper dealing with the many considerations involved in the design and construction of pile foundations in permafrost. With the increased development of the northern areas of the world, it is imperative that our knowledge of this important subject be expanded.

From a construction point of view, permafrost can prove to be an excellent foundation material if dealt with properly. It can also cause considerable grief if some of its properties are overlooked.

Of particular interest in this paper are the graphs that illustrate how the tangential adfreeze strength varies greatly with permafrost temperature and permafrost moisture content. Values given vary from 10 psi to 500 psi.

Fortunately a high adfreeze strength is not essential in normal design. For example, a 20 psi design value for adhesion permits a loading of approximately 4 1/2 tons per linear ft of embedment in permafrost for a 12 in-dia pile.

Where laboratory tests or pile loading tests are carried out to determine a design value for the tangential adfreeze strength, the "creep" or plastic flow characteristics of permafrost must not be overlooked. Given in Table 1 are the results of tests for creep in shear made on frozen-in rods, by Vialov and Skibitzky.[4] The table shows how the load required to produce failure decreases with the time of application of the load. Due to the many uncertainties involved in establishing a safe tangential adfreeze strength, some authorities recommend a factor of safety of 6.

TABLE 1.—CREEP IN SHEAR

Ult. load in percentage of ultimate instantaneous load	100	50	30	25	20	13	10	8.5
Time before failure	9 sec.	3 min.	5 hrs.	4 hrs.	24 hrs.	140 hrs.	766 hrs.	No failure in 4 years

Of particular importance is a complete soil-boring program with undisturbed diamond drill cores taken. In lieu of the usual wash water, kerosene, or other suitable coolant is used during below freezing weather. During summer months compressed air is often used instead of wash water.

The author has demonstrated that where local timber is available, round timber piles driven into pockets of thawed ground, which subsequently refreezes, can very often provide an excellent solution to a foundation problem. Certainly a cost-in-place of $30.00 for a 20-ft spruce pile is very attractive.

Such piles are sometimes driven with the butt down to help prevent the pile from floating up. This has the disadvantage of restricting the carrying capacity to the bearing value of the pile tip. As shown in the author's Fig. 9, a better solution is to drive the pile with the butt up and to wedge the top of the pile if necessary.

According to the author, the only disadvantages found with piles placed by the steam jetting method is that at times piles could not be driven to their full depth due to boulders and also that the boulders sometimes deflected the piles out of position by a few feet.

[2] Pres., Foundations of Canada Engr. Corp. Ltd., Montreal, Canada.
[3] Chf. Structural Engr., Foundations of Canada Engrg. Corp. Ltd., Montreal, Canada.
[4] 4th Internatl. Conf. on Soil Mechanics.

In the case of heavily loaded important structures, this could be a considerable disadvantage. For structures of this type at Frobisher, Northwest Territory, Canada, where the permafrost consisted of sand, silt and gravel, to be certain that all piles would be in their correct location and to their required depth, it was decided to use square timbers in drilled holes. Churn drilling of the holes was called for where heavy gravel or boulders made it difficult or impossible to auger. With this arrangement there is also considerable flexibility in that the cross-sectional area of the piles can be varied to suit loading conditions. Due to the uniformity of the pile cross section, the size of the drilled holes can be kept to a minimum. In this particular case, the piles were 10-in.-by-10 in. with 23 ft of penetration and with a maximum design load of 40 tons. The piles were backfilled with medium sand which was compacted and saturated.

As shown in Fig. 5 of the paper, medium sand with grains approximately 1 mm in diameter has the greatest adfreeze strength. Consequently, such sand is very desirable as fill material around the piles within permafrost. Pile placing was carried out during the winter of 1958-59. Piles were usually in clusters of 3 or 4 piles and were capped with concrete during the milder weather.

Approximately 75% of the holes were drilled with a power auger and the remainder partly by churn drill. At the beginning of operations, the rate of progress with the auger was approximately 12 ft of hole per hour. After 2 mos. an average of 23 ft of hole per hr was achieved. Churn drilling was used when boulders made progress with the auger impossible or very slow. The churn drill achieved approximately 3-1/2 ft of hole per hr. Drilling time with either machine excludes time for moving and setting up. At times during drilling operations, there was considerable damage to the auger's carbide inserts.

ADDITIONAL READING REFERENCES

1. "Arctic Engineering," U. S. Navy, Bur. of Yards and Docks.
2. "Arctic Construction," Depts. of the Army and Air Force, (U.S.).

LINEARLY VARIABLE LOAD DISTRIBUTION ON A RECTANGULAR FOUNDATION[a]

Discussion by N. N. Ambraseys

N. N. AMBRASEYS,[1] M. ASCE.—Formulas and graphs have been developed by Mr. Stamatopoulos to facilitate the computation of vertical stresses and surface displacements under a rectangular flexible foundation caused by a linearly varying surface load distribution.

The writer would like to put forward a solution which he has derived for the problem investigated in the first part of the author's paper, but for all stresses at a point below the unloaded corner of the rectangular foundation. These stresses are given by

$$\sigma_x = \frac{kz}{2\pi}\left\{2\ln\frac{t\,(m+s)}{m+r} - \frac{mn^2}{rt^2} = m\,(1-2\mu)\,\ln\frac{1+r}{1+s}\right\} \quad \ldots\ldots (1)$$

$$\sigma_y = \frac{kz}{2\pi}\left\{m\,\frac{s-r}{sr} + \ln\frac{t\,(m+s)}{m+r} + m\,(1-2\mu)\ln\frac{1+r}{1+s} + (1-2\mu)\ln\frac{m+r}{t\,(m+s)}\right\} \quad \ldots (2)$$

$$\sigma_z = \frac{kz}{2\pi}\left\{m\,\frac{rt^2-s}{rst^2}\right\}, \quad \ldots\ldots (3)$$

which is the author's Eq. 4

$$\tau_{xz} = \frac{kz}{2\pi}\left\{\frac{\pi}{2} - \frac{mn}{rt^2} = \tan^{-1}\left(\frac{r}{mn}\right)\right\} \quad \ldots\ldots (4)$$

$$\tau_{zy} = \frac{kz}{2\pi}\left\{1 - \frac{tr+st-sr}{str}\right\} \quad \ldots\ldots (5)$$

$$\tau_{yx} = \frac{kz}{2\pi}\left\{n\left(\frac{t-r}{tr}\right) - (1-2\mu)\left[m.\,\tan^{-1}\left(\frac{n}{m}\right) + m.\,\tan^{-1}\left(\frac{mr}{n}\right) + \ln\frac{n+r}{s\,(t+n} - \frac{m\pi}{2}\right] + \ln\frac{s\,(n+t)}{n+r}\right\} \quad \ldots\ldots (6)$$

in which $r = \left(1+m^2+n^2\right)^{1/2}$, $s = \left(1+m^2\right)^{1/2}$, $t = \left(1+n^2\right)^{1/2}$ with $m = b/z$, $n = a/z$, and $k = q/a$.

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[a] December, 1959, by A. C. Stamatopoulos.

[1] Imperial College of Science and Technology, Univ. of London, London, S.W. 7, England.

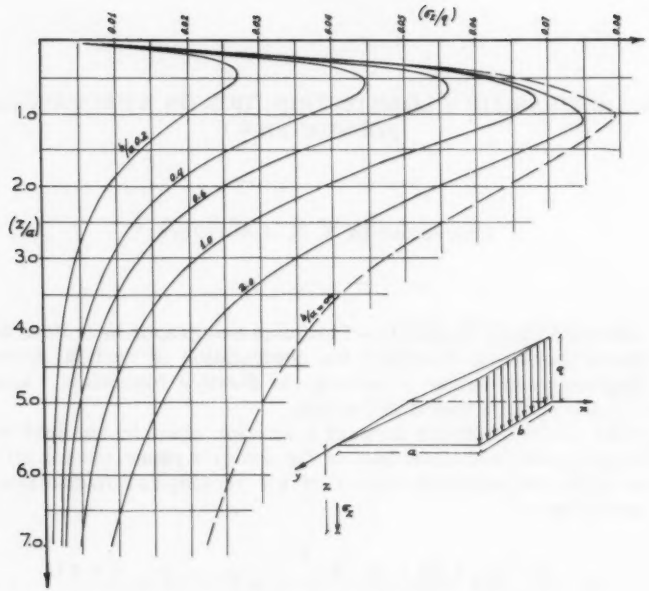

FIG. 1.—VARIATION OF NORMAL STRESS

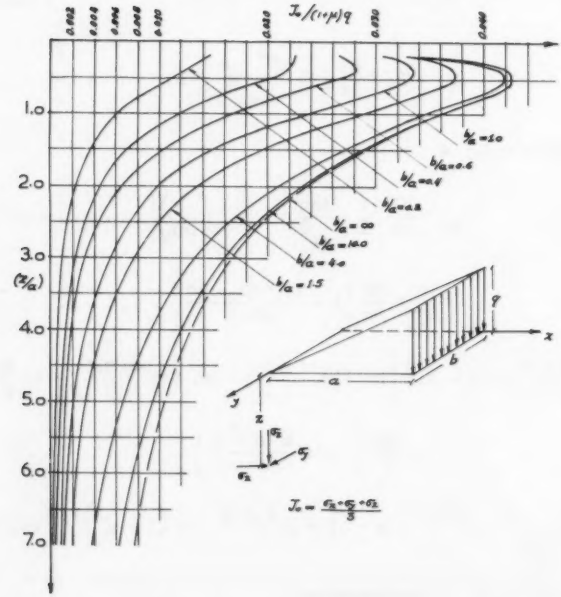

FIG. 2.—VARIATION OF STRESS INVARIENT

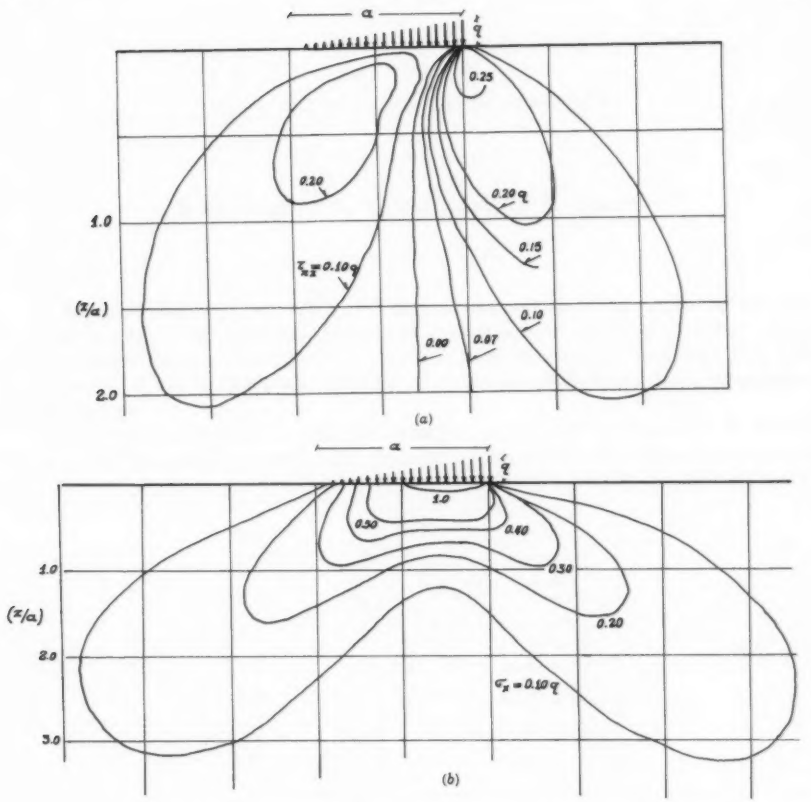

FIG. 3(a),(b).—CURVES OF EQUAL RATIOS FOR A STRIP WITH LINEARLY DISTRIBUTED PRESSURE

The writer has also prepared graphs for the variation of normal stress, σz, with depth under the unloaded corner of the foundation (Fig. 1) and also for the variation of the stress invarient

$$J_o = 1/3 \left(\sigma_x + \sigma_y + \sigma_z\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots (7)$$

under the same point (Fig. 2).

For an infinitely long strip ($b \to \infty$), Figs. 3(a) and 3(b) show curves of equal ratio between a) shear stress τ_{xz}, b) σ_x, and maximum unit load q, on a strip with linearly distributed pressure.

The writer wishes to comment the author on his interesting paper.

DYNAMIC TESTING OF PAVEMENT[a]

Discussion by R. K. Bernhard, D. F. Coates

R. K. BERNHARD,[1] F. ASCE.—The authors are to be commended for presenting a survey of dynamic testing of pavements in the Netherlands. Their paper constitutes an example of a type of nondestructive testing that has been used successfully in the United States as early as 1938, when first presented[2] by the writer.

It is indeed gratifying to note that many results and conclusions of the authors coincide rather closely with findings of american investigators applying

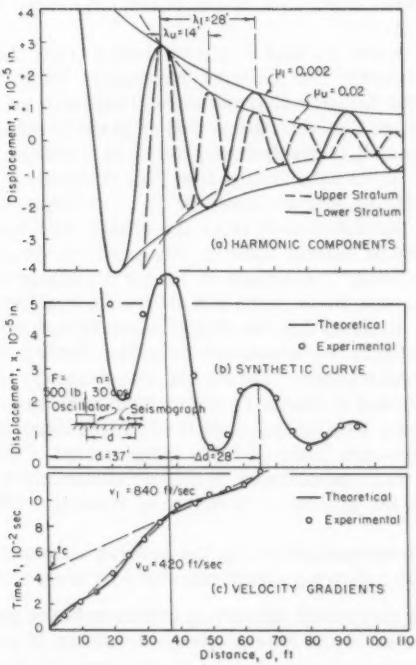

FIG. 1.—DISTANCE VERSUS TIME AND DISPLACEMENT GRAPHS. STRATIFIED SOIL.

[a] February, 1960, by W. Heukelom and C. R. Foster.

[1] Prof., Soil Dynamics Labs., Rutgers, the State Univ. of New Jersey, New Brunswick, N. J.

[2] "Highway Investigations by Means of Induced Vibrations," Engrg. Experiment Sta. Series Bulletin No. 49, Vol. XXXIII, The Pennsylvania State College, 1939.

similar techniques to produce vibratory forces and measuring their effect on the pavement and in the soil.

Critical studies including vibratory-force-producing machines along these lines have been presented by F. J. Converse.[3]

In the authors paper, Fig. 15 is of particular interest. It indicates the straight line relationship of wave length versus distance graph (exciter frequency 45 cps) with characteristic breaking points between the two velocity gradients of 474 fps (144 m per sec) and 826 fps (252 m per sec). These two velocities are almost identical with 420 fps and 840 fps Fig. 1(c) shows a breaking point at a distance of 37 ft from the exciter operating at 30 cps.[4]

Velocity gradients have been used by the discusser to determine strata depths, harmonic components and decay coefficients.

Finally, the lower range for the dynamic moduli of elasticity, starting from approximately 12,000 psi, Fig. 16 is similar to the value of 12,200 psi previously reported.[4]

A comprehensive bibliography[5] of publications on Soil Dynamics can be found elsewhere.

D. F. COATES.[6]--The authors are to be congratulated on the large amount of work that they have done in treating pavement design as a dynamic rather than a static problem.

The conclusion, however, that might be drawn from the paper is that some substantiation is provided for treating pavement design as a static problem. The authors state "the dynamic E-modulus derived from velocity measurements on homogeneous soil tallies with the result of plate bearing tests." The writer interprets this to mean that the dynamic modulus of deformation is the same as the static modulus. It is also stated that "for ϕ equals 0 the traffic stiffness T must be equal to the elastic stiffness R. . . . Consequently, the stresses and the strains can be calculated only from the elastic stiffness R, and the use of conventional theories on elastic constructions, which do not account for mass and damping, is allowed." Although it is not definitely stated that the phase angle could be expected to be zero for all traffic loadings, the writer interprets this statement to mean that the deformation of the subgrade under traffic loads can be more accurately appraised by static testing than by the dynamic testing described in this paper. Hence the two dynamic factors that are considered by this work (and it should be noted that there are others)—the variation of strain with time rate of application of stress—and the inertia effects of a subgrade with mass—are claimed to be insignificant. Consequently, the conclusion, "One of the main advantages of this nondestructive method is its close correspondence with the dynamic character of moving traffic." does not seem to follow.

The writer may be misunderstanding the authors' comments and would therefore be grateful if the following questions could be answered:

1. In view of the increased amount of evidence being gathered that time is an important parameter in the stress-strain relations of many structural ma-

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3 "Compaction of Sand at Resonant Frequency," Symposium on Dynamic Testing of Soils, ASTM, Special Tech. Publication, No. 156, 1953, pp. 124-137.

4 "Microseismic," ASTM Special Technical Publication No. 206, pp. 83 to 102, 1957 and "A Study of Soil Wave Propagation," Highway Research Board Proceedings, Vol. 37, 1958, pp. 618-646.

5 "Symposium on Dynamic Testing of Soils," ASTM, Special Tech. Publication, No. 156, 1953, pp. 254-261.

6 Engr., D. F. Coates, Ltd., Cons. Engrs., Ottawa 1, Ont., Canada.

terials have the authors data which indicates, particularly at the high strain rates involved in pavements, that this is not the case for pavements and subgrades?

2. Do the authors consider that the vibration testing of pavements provides a "close correspondence with the dynamic character of moving traffic" considering the difference in the cases of a vibrating system and a impact system. The steady state response is measured in the one case whereas the transient response to a short period impact would be of interest in the other case.

3. If the statement that the traffic stiffness is equal to the elastic stiffness is true, would this not mean that the vibrating system does not give a "good correspondence" to the impact system?

ERRATA

Journal of the Soil Mechanics and Foundations Division

Proceedings of the American Society of Civil Engineers

February, 1960

p. 85. In Eq. 14a the term 10 S_c / H should be changed to $10^{SC/H}$.

p. 85. In Eq. 25a the term 10 S_c / H should be changed to $10^{SC/H}$.

p. 85. In Eq. 30a the term $10^{S_c/H}$ should be changed to $10^{SC/H}$.

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Department of Conditions of Practice are identified by the symbols (PP). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper number are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 2270 is identified as 2270(ST9) which indicates that the paper is contained in the ninth issue of the Journal of the Structural Division during 1959.

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JULY: 2079(HY7), 2080(HY7), 2081(HY7), 2082(HY7), 2083(HY7), 2084(HY7), 2085(HY7), 2086(SA4), 2087 (SA4), 2088(SA4), 2089(SA4), 2090(SA4), 2091(EM3), 2092(EM3), 2093(EM3), 2094(EM3), 2095(EM3), 2096 (EM3), 2097(HY7)[c], 2098(SA4)[c], 2099(EM3)[c], 2100(AT3), 2101(AT3), 2102(AT3), 2103(AT3), 2104(AT3), 2105(AT3), 2106(AT3), 2107(AT3), 2108(AT3), 2109(AT3), 2110(AT3), 2111(AT3), 2112(AT3), 2113(AT3), 2114(AT3), 2115(AT3), 2116(AT3), 2117(AT3), 2118(AT3), 2119(AT3), 2120(AT3), 2121(AT3), 2122(AT3), 2123(AT3), 2124(AT3), 2125(AT3).

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DECEMBER: 2271(HY12)[c], 2272(CP2), 2273(HW4), 2274(HW4), 2275(HW4), 2276(HW4), 2277(HW4), 2278 (HW4), 2279(HW4), 2280(HW4), 2281(IR4), 2282(IR4), 2283(IR4), 2284(IR4), 2285(PO6), 2286(PO6), 2287 (PO6), 2288(PO6), 2289(PO6), 2290(PO6), 2291(PO6), 2292(SM6), 2293(SM6), 2294(SM6), 2295(SM6), 2296 (SM6), 2297(WW4), 2298(WW4), 2299(WW4), 2300(WW4), 2301(WW4), 2302(WW4), 2303(WW4), 2304(HW4), 2305(ST10), 2306(CP2), 2307(CP2), 2308(ST10), 2309(CP2), 2310(HY12), 2311(HY12), 2312(PO6), 2313(PO6), 2314(ST10), 2315(HY12), 2316(HY12), 2317(HY12), 2318(WW4), 2319(SM6), 2320(SM6), 2321(ST10), 2322 (ST10), 2323(HW4)[c], 2324(CP2)[c], 2325(SM6)[c], 2326(WW4)[c], 2327(IR4)[c], 2328(PO6)[c], 2329(ST10)[c], 2330 (CP2).

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JANUARY: 2331(EM1), 2332(EM1), 2333(EM1), 2334(EM1), 2335(HY1), 2336(HY1), 2337(EM1), 2338(EM1), 2339(HY1), 2340(HY1), 2341(SA1), 2342(EM1), 2343(SA1), 2344(ST1), 2345(ST1), 2346(ST1), 2347(ST1), 2348(EM1)[c], 2349(HY1)[c], 2350(ST1), 2351(ST1), 2352(SA1)[c], 2353(ST1)[c], 2354(ST1).

FEBRUARY: 2355(CO1), 2356(CO1), 2357(CO1), 2358(CO1), 2359(CO1), 2360(CO1), 2361(PO1), 2362(HY2), 2363(ST2), 2364(HY2), 2365(SU1), 2366(HY2), 2367(SU1), 2368(SM1), 2369(HY2), 2370(SU1), 2371(HY2), 2372(PO1), 2373(SM1), 2374(HY2), 2375(PO1), 2376(HY2), 2377(CO1)[c], 2378(SU1), 2379(SU1), 2380(SU1), 2381(HY2)[c], 2382(ST2), 2383(SU1), 2384(ST2), 2385(SU1)[c], 2386(SU1), 2387(SU1), 2388(SU1), 2389(SM1), 2390(ST2)[c], 2391(SM1)[c], 2392(PO1)[c].

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APRIL: 2428(ST4), 2429(HY4), 2430(PO2), 2431(SM2), 2432(PO2), 2433(ST4), 2434(EM2), 2435(PO2), 2436 (ST4), 2437(ST4), 2438(HY4), 2439(EM2), 2440(EM2), 2441(ST4), 2442(SM2), 2443(HY4), 2444(ST4), 2445 (EM2), 2446(ST4), 2447(EM2), 2448(SM2), 2449(HY4), 2450(ST4), 2451(HY4), 2452(HY4), 2453(EM2), 2454 (EM2), 2455(EM2)[c], 2456(HY4)[c], 2457(PO2)[c], 2458(ST4)[c], 2459(SM2)[c].

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JUNE: 2494(IR2), 2495(IR2), 2496(ST6), 2497(EM3), 2498(EM3), 2499(EM3), 2500(EM3), 2501(SM3), 2502 (EM3), 2503(PO3), 2504(WW2), 2505(EM3), 2506(HY6), 2507(WW2), 2508(PO3), 2509(ST6), 2510(EM3), 2511 (EM3), 2512(ST6), 2513(HW2), 2514(HY6), 2515(PO3), 2516(EM3), 2517(WW2), 2518(WW2), 2519(EM3), 2520 (PO3), 2521(HY6), 2522(SM3), 2523(ST6), 2524(HY6), 2525(HY6), 2526(HY6), 2527(IR2), 2528(ST6), 2529 (HW2), 2530(IR2), 2531(HY6), 2532(EM3)[c], 2533(HW2)[c], 2534(WW2), 2535(HY6)[c], 2536(IR2)[c], 2537(PO3)[c], 2538(SM3)[c], 2539(ST6)[c], 2540(WW2)[c].

c. Discussion of several papers, grouped by divisions.

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PART 2

JUNE 1960 — 21 NO. SM 3
VOLUME 86 PART 2

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NEWS OF THE SOIL MECHANICS AND FOUNDATIONS DIVISION OF ASCE

JOURNAL OF THE SOIL MECHANICS AND FOUNDATIONS DIVISION
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DIVISION ACTIVITIES

SOIL MECHANICS AND FOUNDATIONS DIVISION

Proceedings of the American Society of Civil Engineers

NEWS

June, 1960

NEWS FROM THE WEST COAST

The trials and tribulations of dam design and construction have captured the imagination and interest of California where population growth has intensified the water supply problem and where a multi-billion dollar bond issue confronts the voters this fall. Over the past few months the technical meetings have reflected this increased interest in dams and related matters. The failure of the Malpasset Dam in France has been discussed both by engineers and engineering geologists who visited the site and studied the results of the failure. On January 19, J. Barry Cooke of the Pacific Gas and Electric Company described his engineer's point of view to a joint meeting of the Hydraulics and Power Divisions of the San Francisco Section, then followed on March 1 with a talk to the Structural Engineers Association of Northern California. In between these dates, Dr. F. A. Nickell presented a geologist's view to the California Association of Engineering Geologists, a new organization with sections in Los Angeles, San Francisco and Sacramento. Dr. Nickell also commented on and showed colored slides of the diversion tunnel gate chamber failure of the Bhakra Dam, in India.

During April and May the University of California at Berkeley has presented a special series of lectures on the "Practical Aspects of Earth Dam Design and Construction" which were given by Dr. J. L. Sherard of Woodward, Clyde, Sherard and Associates. Engineers of the Bay Area had been invited to attend.

The University has also presented a series of two lectures by Professor E. C. W. A. Geuze of the Delft Technological University:

On Monday, April 4 - "The Effect of Time on the Shearing Strength of Clay"
On Tuesday, April 5 - "Analysis and Design of Pile Foundations in the Netherlands"

Again the University kindly extended an invitation to engineers of the Day Area to attend.

In Los Angeles, at a joint meeting of the local Soil Mechanics and Foundations Division and the California Association of Engineering Geologists, Travis Smith of the Materials and Research Department, California Division of Highways, spoke on "Landslides and Horizontal Drains." Use of these as a means of combating landslides has become very popular in the area.

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At a meeting on March 30 of the Air Transport Division of the San Francisco Section, Ben Nutter, Chief Engineer for the Port of Oakland, spoke on the present expansion of the Oakland (California) Airport which involved reclamation of tidal flat land from San Francisco Bay for a new 10,000 foot jet runway and the first increment of a new passenger terminal building.

SOIL MECHANICS WORKSHOP ON SOIL CLASSIFICATION AND COMPACTION

A one-day Soil Mechanics Workshop on Soil Classification and Compaction was conducted at the University of Cincinnati on February 12, 1960. It was co-sponsored by the Soil Mechanics and Foundations Division, Cincinnati Section and the Department of Civil Engineering, University of Cincinnati.

The Program included the following:

Geological Origin of Local Soils
Subsurface Explorations
Soil Classification Tests
Soil Classification Systems
The Unified Soil Classification System
Preparation and Use of Boring Logs
Basic Moisture Density Relationships of Remoulded Soils
Methods of Specifying Soil Compaction
Compacted Earth Fill Construction
Evaluation of Base Course and Fill Construction
Special Problems in Fill Construction

The Workshop Program was arranged for design and construction engineers who had had no formal training in Soil Mechanics, and was designed to promote a better understanding of some limited applications. It was well attended and generally well received by the 130 registrants. About 40 who answered questionnaires after the Workshop indicated a desire for further presentation of information of Soils by this means. The next topic preferred was Foundation Design. Tentative plans are being made to hold such a Workshop in the Fall of 1960.

INDIAN SYMPOSIUM ON FOUNDATION ENGINEERING

A symposium on foundation engineering sponsored by the Indian Institute of Science will be held at Bangalore, India during November of this year. The exact dates for the symposium have not yet been established, but papers are being solicited by the sponsoring group. Papers on any of the aspects of foundation engineering may be submitted. A broad classification of the proposed symposium is given below:

Exploration and Field Investigations
Theory and Design of Foundations
Pile Foundations
Foundations Other Than Piles
Foundations Other Than Piles

Foundation Treatment
Machine Foundations
Miscellaneous

Short synopses of proposed papers should be submitted by May 31, 1960. Papers are to be submitted by September 30, 1960. All correspondence relative to papers or additional information concerning the symposium should be sent directly to:

Mr. C. N. Nagaraj
Civil and Hydraulic Engineering Section
Department of Power Engineering
Indian Institute of Science
Bangalore 12, India

ARRANGEMENTS FOR CONVENTION PROGRAMS

The technical program sessions for the Soil Mechanics and Foundations Division at the three national conventions of the Society in 1960 are being arranged under the supervision of T. M. Leps, Vice Chairman of the Executive Committee. To encourage effective local contributions and participation, it has been the practice this year to appoint a Local Chairman for program arrangements. For the New Orleans Convention which was so successfully executed, Mr. J. Bres Eustis of the firm of Eustis Engineering Company, was Local Chairman. For the Reno Convention in June, Mr. A. W. Root of the California Division of Highways is Local Chairman, and Messrs. S. J. Johnson and P. T. Bennett have been active in its planning. For the Boston Convention in October, Dr. Harl P. Aldrich of the firm of Haley and Aldrich, is Local Chairman. He has arranged locally for the help of a committee composed of Arthur Casagrande, W. F. Swiger, T. W. Lambe and James Haley. Their joint efforts promise an outstanding program which is planned to be given in four half-day sessions.

The efforts and thinking of these people, dedicated to the design of improved programs for the dissemination of engineering knowledge, are greatly appreciated and represent an excellent opportunity for individual members to contribute in a most productive manner to the advancement of our Society.

Mr. R. A. Barron of the Executive Committee will be in charge of the Division's program planning for 1961 conventions. Offers of assistance in arranging the programs at Phoenix and New York may be addressed to him.

VISITING LECTURER FROM DELFT TECHNOLOGICAL UNIVERSITY

Professor Ir. E. C. W. A. Geuze, c.i., made his first visit to the United States between February 25 and April 27, 1960. He presented a series of lectures at Rensselaer Polytechnic Institute, presented a paper entitled "The Effect of Time on the Shearing Strength of Clays," at the New Orleans Convention of the A.S.C.E., and visited various universities and soil mechanics laboratories throughout the United States.

In addition to being a Visiting Professor of Soil Mechanics at Rensselaer Polytechnic Institute, Profess Geuze's official titles are Professor of Civil Engineering, Delft Technological University, Delft, The Netherlands, and Director of the Delft Soil Mechanics Laboratory, Hydraulic Engineering Laboratory Foundation, Delft, The Netherlands.

A summary of Professor Geuze's itinerary follows:

February 26 - March 6	Rensselaer Polytechnic Institute
March 6 - 10	New Orleans A.S.C.E. Convention
March 10 - 12	Waterways Experiment Station
March 13 - 16	Purdue University
March 16 - 18	University of Illinois
March 18 - 22	Office of Naval Research
March 22 - 25	M.I.T., Harvard, Boston Society of C.E.
March 25 - 27	Rensselaer
March 27 - 30	U. S. Bureau of Reclamation, Denver
March 30 - April 2	Northwestern University
April 2 - 6	University of California, Berkeley, U. S. Army Engr. Div. Laboratory, Sausalito
April 6 - 10	Princeton University
April 11 - 12	University of Florida
April 13 - 15	University of Texas
April 15 - 20	Columbia University
April 20 - 27	Rensselaer Polytechnic Institute

Four topics on which Professor Geuze spoke are:

"Analysis and Design of Pile Foundations in the Netherlands"
"Ground Water Control in the Netherlands"
"The Effect of Time on the Shearing Strength of Clay"
"Physio-Chemistry of Soil as an Important Asset to the Understanding of Their Mechanical Properties"

These talks formed the basis for many interesting seminars held in conjunction with Professor Geuze's visit to our country.

OFFICIAL REGISTER CORRECTION

The Official Register for 1960 contains an incorrect listing of the members of the Task Committee on Cement Grouting of Rock Foundations of the Division's Committee on Grouting. The roster of this Committee is:

Judson P. Elston, Chairman
A. Warren Simonds
W. K. Seaman
F. H. Lippold
F. C. Sturges
*J. M. Polatty
*A. E. Rose
*B. E. Clark

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* Serve in an advisory capacity.

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AUGUST NEWSLETTER

Deadline date for arrival at this office of contributions for the August Newsletter: June 28, please.

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